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ECONOMIC & REGIONAL DEVELOPMENT AGREEMENT

Final Report
TASK 1A - VBM Design Guidelines
Chapter 3 - VBM Stope Design

by

Inco Limited, Thompson Division
Thompson, Manitoba

March 1990

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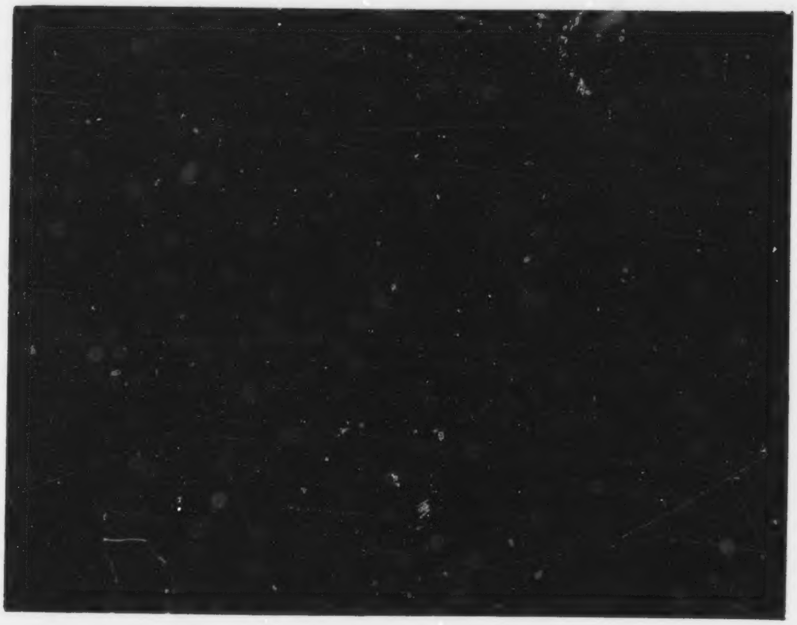

CANADA - MANITOBA

MINERAL DEVELOPMENT AGREEMENT

RESEARCH & DEVELOPMENT IN MANITOBA MINES

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RESEARCH & DEVELOPMENT IN MANITOBA MINES

VERTICAL BLOCK MINING (VBM) DESIGN GUIDELINES

Chapter 3

VBM Stope Design

This chapter has been prepared as part of the

Task 1
To Develop Vertical Block Mining Design Guidelines
For A Steeply Dipping Orebody;
And To Develop Scaling Techniques
of the
Canada/Manitoba Mineral Development Agreement

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MINERAL DEVELOPMENT AGREEMENT

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VERTICAL BLOCK MINING (VBM) DESIGN GUIDELINES

The Vertical Block Mining (VBM) Design Guidelines consists seven chapters, published separately. The seven chapters are:

Summary

1. **Cut and Fill Mining**
2. **Data Collection Systems**
3. **VBM Stope Design**
4. **Drilling and Blasting**
5. **Support Requirements**
6. **VBM Stoping Procedures**

Reference to this chapter should be quoted as follows: "**Vertical Block Mining Design Guidelines Chapter 3 - VBM Stope Design**"; Canada/Manitoba Mineral Development Sector B - Mining; DSS File No. 14SQ.23440-4-9147-3; CANMET Project No. 4-9147-3; March 1990

EXECUTIVE SUMMARY

Chapter 3: VBM Stope Design

A major factor in the planning of an underground mine is the choice of suitable excavation dimensions. The economy of size suggests that the larger the excavations, the lower the costs. As larger mining equipment can be used, a reduced work force is required. The excavation size, along with the ore geometry, is used to determine the most suitable mining sequence.

A philosophy of stope design has been developed in which past experience is used to calibrate both empirical and numerical models. In turn, these models are used to assess applicable stope dimensions and to predict ground control events associated with different stoping sequences. This must be done within the frame work of ore value, [illegible]nnage and prediction requirements.

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Chapitre 3: Design des chantier VBM

Les principal facteur dans la planification d'une mine souterraine est le dimensionnement optimal des chantiers. Le dimensionnement économique suggère que pour de plus grandes excavations, il y aura de plus faibles coûts de production. Dans ce cas, de plus gros équipements miniers peuvent être utilisés donc la force de travail requise est réduite. La dimension des excavations et la géometrie du gisement sont utilisées pour déterminer la séquence de minage la plus propice.

La philosphie du design des chantiers a été développée en utilisant les expériences précédentes pour calibrer les modèles numériques et empériques. A tour de rôle, les modèles sont utilisés pour évaluer la dimension des chantiers et pour prédire le contrôle de terrain associé aux différentes séquences de minage. Ceci doit être fait initialement avec le calcul de la valeur du gisement, du tonnage et des prédictions requises.

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EXECUTIVE SUMMARY

Chapter 3: VBM Stope Design

A major factor in the planning of an underground mine is the choice of suitable excavation dimensions. The economy of size suggests that the larger the excavations, the lower the costs. As larger mining equipment can be used, a reduced work force is required. The excavation size, along with the ore geometry, is used to determine the most suitable mining sequence.

A philosophy of stope design has been developed in which past experience is used to calibrate both empirical and numerical models. In turn, these models are used to assess applicable stope dimensions and to predict ground control events associated with different stoping sequences. This must be done within the frame work of ore value, tonnage and prediction requirements.

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Chapitre 3: Design des chantier VBM

Les principal facteur dans la planification d'une mine souterraine est le dimensionnement optimal des chantiers. Le dimensionnement économique suggère que pour de plus grandes excavations, il y aura de plus faibles coûts de production. Dans ce cas, de plus gros équipements miniers peuvent être utilisés donc la force de travail requise est réduite. La dimension des excavations et la géometrie du gisement sont utilisées pour déterminer la séquence de minage la plus propice.

La philosphie du design des chantiers a été développée en utilisant les expériences précédentes pour calibrer les modèles numériques et empériques. A tour de rôle, les modèles sont utilisés pour évaluer la dimension des chantiers et pour prédire le contrôle de terrain associé aux différentes séquences de minage. Ceci doit être fait initialement avec le calcul de la valeur du gisement, du tonnage et des prédictions requises.

1.0 OBJECTIVES

The purpose of this chapter is to design the optimal VBM stope in terms of size by evaluating such parameters as the characteristics of the rock mass, and using empirical and numerical models thereby eliminating or reducing to the minimum potential stability problems.

2.0 INTRODUCTION

The Thompson Mine ore zone was discovered in 1956 and production commenced in 1961. The ore zone extends 5.6 km (3.5 miles) and is serviced by the T-1 and T-3 shafts. The shafts are 3.2 km (2 miles) apart and are connected on several levels. The T-1 shaft at the south end is the main production shaft, while the T-3 is a service shaft for the north end of the mine. The deepest operating levels at T-1 and T-3 are 4000 and 2400 level respectively. A longitudinal section of Thompson Mine is displayed in Fig. 3.1. All the ore from the T-3 section of the mine is conveyed through internal ore passes to the 2400 level where it is trammed by rail to the T-1 main ore pass.

Thompson Mine has been converting from a cut and fill mining to vertical block mining (VBM) since 1981. Approximately 82% of present production comes from VBM, 14% cut and fill and 4% undercut pillar stopes. Total daily hoist averages 5443 tons (6000 tons).

In this chapter, it is important to note that open stope mining is used synonymously with vertical block mining. At Thompson Mine, the term stope is defined as a large mining area containing numerous blocks. It is realized that this is opposite to industry terminology, however this can be explained as a result of the conversion from cut and fill to vertical block mining.

There are two types of open stope mining currently used at Thompson Mine. In general, the width of the orebody dictates the method. Longitudinal mining is used in narrow areas, where the orebody is less than 9.1m (30 ft.) wide. Transverse mining is the main method in wide zones 9.1m (30 Ft.) or in cases of bad ground in narrow areas, which will be discussed later in this chapter. Longitudinal mining is displayed with a typical cross and long section in Fig. 3.2

The advantage to longitudinal mining is that the ore zone limits are defined by the drilling horizon (top sill, top sub) and the mucking horizon (bottom sill, bottom sub) thus reducing to a minimum the reliance on diamond drill holes. Since the majority of development is in ore the overall stoping costs are reduced.

The main disadvantage of longitudinal retreat is that both horizons must be kept open for the mining duration of the stope. With increased mining, stresses are added to the virgin stresses causing back conditions to deteriorate. At some point it may become necessary to perform a complete reconditioning of the back, as was the case of the 1403-301 stope. Another constraint of longitudinal retreat is the committment to retreat mining, thus reducing sequencing flexibility. The most efficient block extraction by longitudinal retreat is to retreat from the ends of the stope to the center. In this manner, one can mine a block at one end while filling the previously mined block at the other end, thus maintaining continuous production. The result is that conditions gradually deteriorate with the creation of a highly stressed final block/pillar. Transverse mining is displayed in Fig. 3.3 with a cross and long section. This variation of open stope mining not only increases productivity because of the larger stope size but also provides flexibility in sequencing since there is no restriction to retreat mining. It also provides a location for diamond drill holes which allow a better delineation of the orebody which in turn permits improved engineering of each block.

The disadvantage is the increase in footwall (and/or Hangingwall) rock development needed to prepare the stope for mining. The approximate cost of development is 350 to 500 dollars per foot depending on the size of the drift.

Fig. 3.4 exhibits a plan view with the 1803-330 transverse stope to the left and the 1803-331 longitudinal stope to the right. The ore zone is outlined by the dotted line. The transverse section displays the extensive footwall rock development needed to access the thicker part of the orebody. In contrast the longitudinal section consists of a single drift development following the ore outline along the entire strike length.

3.0 OPEN STOPE DESIGN CONSTRAINTS AND PROBLEMS

3.1 Typical Canadian Mine Open Stope Dimensions

A study by Pakalnis (13)(1986) presented the range of open stope dimensions typically found in Canadian Mines. The results of this study provide a framework against which the stope dimensions at Thompson can be compared.

Thompson Mine stope dimensions in comparison with Canadian open stopes are smaller than average, as displayed in Fig. 3.5. From the Canadian open stope database for stope spans, 60% of the stopes are longer than 27m (90 ft.) and 50% of those are longer than 61m (200 ft.) For stope heights, a full 48% of the database stopes are greater than 61m (200 ft.) and 48% are situated between 30.5m (100 ft. and 61m (200 ft.) range. For Canadian stope widths 24% of the mines studied had widths greater than 20m (65 ft.) and 12% had widths in the range of 15 to 20m (45 to 65 ft.) Over 50% of the Canadian stopes have a hydraulic radius larger than 10 and over 90% of the stopes are included in the greater than 5 range.

3.2 Typical Thompson Mine Open Stope Dimensions

- **Stope Height**

Typical stope heights along dip for VBM blocks at Thompson Mine is shown in Fig. 3.6. The average is 40m (130 ft.) The height can vary depending on factors such as the dip, the location of existing levels, the number of sublevels developed, and the relative elevation of the cut and fill stope prior to conversion. Where the geological outlines are inconsistent or indicate a narrow ore zone, block heights are reduced in order to initiate selective mining.

- **Stope Length**

Fig. 7 displays the histogram of the stope strike lengths at Thompson Mine. The average is 18m (60 ft.) for this dimension. Longitudinal blocks have been mined with lengths up to 135 feet. The shorter lengths are indicative of transverse mining blocks. Block lengths are established by experience for each stope in an attempt to reduce wall rock dilution.

- **Block Width and Ore Dip**

A histogram of both the typical block width reflecting the local thickness of the orebody and the average dip are shown in Figures 8 and 9 respectively. The average block width is 9.1m (30 ft.) and the average dip is 69 degrees.

- **Shape Factor**

The shape factor S is the hydraulic radius of the particular surface being analyzed which is defined as the area divided by the perimeter in meters.

The hangingwall shape factors of Thompson Mine blocks are presented in Fig. 3.10. The shape factor average being 5.8. Comparing this value to Canadian standards, it situates itself on the bottom end of the scale.

Thompson Mine stope back shape factors average 2.6, as displayed in Fig. 3.11 for individual blocks. As mining advances and more blocks are extracted, the back shape factor for a stope increases only slightly to an average of 3.1 as shown in Fig. 3.12. The low S value reflects the long and narrow shape of the backs which the hydraulic radius favours over squarer type surfaces.

- **Tons Removed**

The histogram in Fig. 3.13 shows the tons removed for the 126 blocks in the database. Over 82% of the blocks vary between 4,535 and 22,670 tonnes. At Thompson Mine block tonnages are measured using the scooptram bucket count from the drawpoint pull. This method has been found to be the most accurate in comparison to conventional measures, such as, the tons of fill placed or visual inspection of the block after mining.

From the calculated average of 126 blocks at Thompson Mine, the typical open stope dimensions (Fig. 3.14) are 39.6m (130 ft.) in height along dip, a width of 9.1m (30 ft.) and a length of 18.3m (60 ft.) along strike. The average dip is 70 degrees. This adds up to an average of 20,865 tonnes per block. The small blocks in comparison to other Canadian mines are due to the weaker ground present at Thompson, notably the schistose hangingwall, and the narrow orebody.

One would expect from the economy of size, that the larger the excavations, the lower the costs. Larger mechanized equipment can be used requiring a reduced workforce. Generally the larger the excavations, the longer the cycle time which in turn directly affects the overall mine productivity. Tons/shift, a measure of productivity, generally increases as the stope size increases. An example is the 1803-330 stope where block 4 and 5 were mined as one block. A total of 58,265 tonnes were removed from this double block in 48 shifts, an average of 1214 tonnes per shift. This amounts to a period of just over one month on a double shift basis. A graph showing the general trend between the level of productivity vs. stope size is displayed in Fig. 3.15. Maximum productivity is obtained in the 54,400 to 59,000 tonnes stope size range. However, it is important to keep in mind to properly assess a block for stability prior to mining to eliminate serious ground control problems which will adversely affect productivity levels due to the limited number of mining blocks in production at any one time.

3.3 Ground Control Problems in Canada

Data relevant to ground control problems in open stoping operations has been obtained for 31 Canadian mines and is displayed in Fig. 3.16. The problems were simply classified into three categories; severe, moderate and little or none.

Referring to Fig. 3.16, about 20% of the mines experienced severe problems with dilution, 60% have moderate problems with dilution. Other ground control problems in the severe category include pillar instability, stope instability and caveability. Of these, the last two can be traced back to dilution as they are intimately related.

With respect to the moderate category, 25% of the mines indicate fragmentation problems. Improper fragmentation is a two fold problem. If large chunks are produced there is a necessity for secondary blasting. If the ore is overfragmentated, the problem lies with the muck not wanting to flow in the drawpoint making it difficult for the scooptram to pick up the muck.

In all, 30% of the mines have indicated moderate problems with drift maintenance. Stope instability has shown to be a problem in 35% of all mines while 40% of the mines indicated drawpoint hang ups as a problem. Stope instability is synonymous to sloughing and when sloughing occurs it is usually in the form of large chunks. These chunks in turn produce drawpoint hang ups.

3.4 Ground Control Problems at Thompson Mine

In discussing the above factors with key personnel the Thompson Mine open stope ground control problems have been rated as displayed in Fig. 3.17.

Severe problems are experienced with dilution and blast damage. Open stope mining has not proven to offer extensive opportunity to control the grade, and as conversion to VBM is advancing, the overall hoist ore grade is reducing. Blast damage is predominant in the narrow stopes. Thompson Mine applied crater or modified crater/slash blasting in narrow orebodies with weak hangingwalls, in order to achieve productivity gains. The trade-off has been an increase in dilution.

The fragmentation at Thompson Mine is a moderate to severe problem. The cratering technique used at Thompson Mine has the effect of pulverizing the rock located in the volume of each crater and producing large chunks from blast damaged back and wall rock. Fig. 3.18 displays fragmentation as measured at the drawpoints of different stopes and illustrates the bimodal nature of the framentation achieved at Thompson. It is evident the dominants of fines and occurrence at larger chunks affects the productivity levels.

In the moderate class, six ground control problems are identified; roof falls, stope instability, caveability, drawpoint hang ups, drift maintenance and redrills. The first four problems are inter related and can best be displayed by the histogram in Fig. 3.19. Hangingwall failures are most common at Thompson Mine and have occurred in 30% of the blocks. The identified cause being the unsupported and weak biotite schist hangingwalls and blast damage. Back failures have occurred in 11% of all cases in the database. Similarly, 9% of all blocks experienced footwall failures.

Drawpoint hang ups vary in severity depending on the size of the chunks. Pieces slightly too big for the ore pass system tend to be ore while extremely large pieces appear to be wall rock due to sloughing.

Redrilling of holes tends to be a localized problem due to the high stress conditions in particular zones. In most cases, holes have been completely squeezed out so that explosives cannot be inserted. In addition, spalling occurs along the wall of the hole and small pieces block access for charging of the hole. The redrilling of this type of hole usually results in the immediate plugging once the drill rods are taken out, therefore, it is better to redrill a completely new hole.

Drift maintenance or reconditioning is predominant in the drilling horizons usually as a result of the application of longitudinal retreat mining. In situations where mining retreats from both ends of the stope a highly stressed pillar and back area is created and deteriorates to a point where the back has to be completely reconditioned.

Other problems that were classified in the little or of no concern were pillar instability, seismic events, subsidence and water.

3.5 Impact of Hangingwall Failure

Displayed in Fig. 3.20 is the major impact of hangingwall failure; dilution. Dilution not only reduces the grade but creates additional muck to haul, tram, hoist and process. In many cases, the hangingwall slough is mineralized but was intended to be left behind and deemed uneconomic. Dilution can be shown to be very costly in dollar terms but, it is a very difficult problem to control.

Hangingwall failure increases the width and length of the block and increases the possibility of inducing back failure. Approximately 50% of the back failures at Thompson Mine are directly related to initial hangingwall failures.

Mining a block above or adjacent to which a hangingwall failure has occurred can be very difficult. The problem lies in that the failure plane extends itself in both directions and is very hard to control. Cable bolting the hangingwall could possibly solve the problem but is not a current practice at Thompson Mine.

As stated in the previous section, large chunks end up in the drawpoint which adversely affects safety and productivity. There have been instances where so much material sloughed that the ore left in the block was judged irrecoverable.

3.6 Impact of Back Failure

The impact of back failure can be summarized by four main points. Firstly, as shown in Fig. 3.21, when a back failure occurs it presents numerous problems for mining of the block directly above. The usual procedure is to sandfill the void created by the failure as an extension to filling the mined block below. When mining eventually proceeds to the block above, all this sand must first be mucked out. The sand is transported by scooptram either to an empty block which is being filled at the time or to an ore pass, thus diluting the mine ore hoist. Sand is not allowed in the waste pass since this pass is not pulled on a regular basis. Equally important, the surveyors find it difficult if not impossible to pick up the breakthrough pattern of the drill holes. Drill holes must be picked up to produce efficient blasts and eliminate the charging of undesirable holes.

Secondly, the creation of large chunks in the drawpoint create mucking difficulties. Although, back failure chunks are usually ore, they present a major problem with respect to their removal. Solving this problem by secondary blasting creates a hazardous situation for the person drilling the chunks. In addition, the mucking process is delayed reducing productivity levels.

Thirdly, as a new block is opened adjacent to a filled block whose back has failed, the exposed span of the back has now doubled and a new failure arch tends to be created, the arch being large enough to cover the new span.

Fourthly, if the mine is running on a tight schedule when unanticipated failure occurs producing large chunks, the concern is concentrated on solving the scheduling problem to satisfy the daily production requirements. Other drawpoints have to increase production to cover for the drawpoint experiencing the hang up.

4.0 THEORY OF OPEN STOPE DESIGN

4.1 Prediction Model

Not until very recently has there been models which mining engineers can utilize to guide them in designing open stopes. Mining engineers still rely heavily on experience from mining in similar conditions and use a trial and error approach. A better solution is to merge the experience with reliable prediction models to gain a better understanding of the rock mass behaviour.

A prediction model is basically an empirical relationship that takes into account the rock mass characteristics, the effect of stress redistribution and the stope span, to estimate the relative stability of an open stope surface. These models are based on existing rock mass classifications systems which have been adapted to open stope mining.

In the mid seventies applied rock mechanics produced two major rock mass classification systems in which the rock mass was separated into quantifiable parameters. These are the RMR-system by Bieniawski (2) (1973) and the Q-system by Barton et al (1) (1974). Both rock mass classification systems are described in Appendix 3A. These two classification systems are oriented towards tunnel design and prove to be extremely conservative when applied to open stopes. Both authors have attempted to adapt their system for mining applications with factors such as the Excavation Support Ratio (ESR) and the stand up time of an excavation. These factors are been overly simplistic and do not overcome the differences between mining and tunnelling.

In 1976, Laubscher (9) (1976) modified the RMR-system for mining applications, though principly applicable towards caving methods of extraction. This classification system is better used by specialists due to the lack of guidelines with respect to the proper selection of adjustment factors.

In 1981, Mathews et al (10) 1981) developed a new empirical approach for predicting the stability of open stopes at depths exceeding 1000 meters. This empirical model relates the depth of mining, the rock mass quality and the stope span. For each given stope surface, a stability number (N) derived from geotechnical parameters is plotted against a shape factor (S).

The stability number accounts for the rock mass quality, the state of stress and the orientation of the exposed surface. It is expressed as;

$$N = Q' * A * B * C$$

where Q' = modified version of the NGF rock mass rating
A = stress factor
B = rock defect orientation factor
C = design surface orientation factor

The shape factor(S) is the calculation of the hydraulic radius of the stope surface.

The state of stability or instability is obtained on the stability graph from the plot of the resulting coordinates on a semi-log graph. Therefore, a high stability number (N) combined with a low shape factor (S) reflects stable exposures while the inverse relation reflects instability. The levels of stability were classified by Mathews into three categories; stable, potentially stable and potentially caving. A thorough review of this empirical relationship has been completed by Potvin (15) (1988) and Greer (4) (1989).

The Mathews method has become the best prediction model for assessing open stope stability. When initially introduced, the empirical relationship had only a small database. Potvin (15) (1988) applied the method extensively and found it very promising but thought that the zones defined on the stability graph were vague for adequate design. Potvin recalibrated Mathews stability graphs using data from over 30 mines in Canada to create a database comprising of 84 case histories. He effectively squeezed the transition zone by modifying the calculation of the stability number (N) in order to better fit the data to the stability graph. The result is a modified stability graph containing only two zones, one indication stable conditions and the other indicating caving conditions. The two zones being separated by a small grey zone representing potential instability.

The work by Mathews and Potvin has enabled better prediction of open stope stability. Fig. 3.22 is an example from Potvin displaying how the various factors relate to the stability graph. A site calibration is necessary in order to gain confidence with the prediction model.

4.2 Flow Sheet Process

A flow sheet of the stope design process is presented in Fig. 3.23 and is described in Sagman and Greer (16) (1986). The procedure is to take a regional geological model and choose a suitable variation of open stope mining for a particular stoping area. The geology department then outlines the orebody geometry along with the varying grades of ore and any rock if present. At the same time, the engineering department conducts line surveys to provide the rock mass characteristics of the ore and host rock.

From this information, a conceptual layout is produced. The layout will guide the engineer in calculating the overall tonnage and grade, or production base figure, of the mining block. Further refinements can be made to optimize the production while always keeping in mind the geological constraints. After finalization of the stoping layout, a ground support system can be designed and implemented.

To assess stope stability of the conceptual layout other than solely by experience, empirical models such as the Mathews method and its derivative from Potvin are used. As described in the previous section, these models take into account the rock mass characteristics which describe the overall quality of the rock mass.

Numerical models are used in parametric studies to give an indication of the induced stresses due to the creation of an opening. These induced stresses define a stress factor which serves in the empirical models.

The resulting analysis will determine the extent of the stability for that particular stope. If no ground problems are anticipated the actual mining plan is implemented. Assuming minor problems, a simple adjustment may be needed before mining proceeds. If major problems are encountered in the stope stability analysis, then a complete redesigning of the stope is required.

Calibration of the model is accomplished by back analysis. Once a stope has been mined out it is an excellent idea to visually inspect the stope in order to signal if any hangingwall slough has occurred, its extent and the ground condition of the stope back. This technique helps to calibrate the prediction model to the mine site instead of being based on an average of Canadian open stopes. This back analysis technique can be defined as the experience curve as displayed in Fig. 3.23.

5.0 ROCK MASS CHARACTERIZATION

5.1 Rock Mass Quality, RQD, and Strength

An example of the Thompson Mine typical lithology is shown in Fig. 3.24. To the left is the predominant schist biotite hangingwall adjacent to the sulphide ore zone. The footwall also shows some localized schist biotite but quartzite is the predominant footwall rock type. Within the sulphide ore zone are localized zones of weak peridotite.

The rock mass quality is expressed by the quality index (Q) which was developed by Barton et al (1) (1974) of the Norwegian Geotechnical Institute. The Q value varies from 0.001 to 1000 with a rock quality assessment varying from exceptionally poor to exceptionally good respectively. The Q value is calculated by the following equation:

$$Q = RQD/Jn * Jr/Ja * Jw/SRF$$

where,
RQD = Rock Quality Designation
Jn = Joint Set Number
Jr = Joint Roughness Number
Ja = Joint Alteration Number
Jw = Joint Reduction Factor

The empirical model chosen uses a Q' value in which the Strength Reduction Factor (SRF) is defaulted to one, therefore, utilizing the first two quotients only.

At Thompson Mine, the quality values obtained for the hangingwall, back and footwall range from fair to good. Although the back tends to have slightly poorer values, all histograms in Figures 3.25, 3.26 and 3.27 oscillate near the Q' value of 10.

Figures 3.28, 3.29 and 3.30 display the histograms of RQD values for Thompson Mine's hangingwall, ore zone and footwall. All values are quite high, with hangingwall and footwall RQD'S averaging 85% while the ore zone shows a slightly higher value of 90%.

Thompson Mine values of the intact rock strength for the hangingwall is 70 MPa which shows the large influence of the weak schistose walls. The ore zone or stope back is either composed of a massive sulphide or a sulphide matrix and has a strength value equal to 72 MPa. The footwall strength is 70 MPa for the schist biotite walls and 130 MPa for the quartzite portion. All values were from laboratory uniaxial compressive strength tests performed by geological consultants.

5.2 Stress Regime

There have been numerous in-situ stress measurements performed at Thompson Mine. Canmet took measurements at the 2000 level while other measurements were recorded at the 400, 1000 and 4000 levels by geological consultants. The results were inconsistent on the 400 level but at greater depth the measurement results clustered. The principal stress orientations were similar for all test sites. The results agree well with Canadian Shield data.

The goal of a stress measurement program is to establish a vertical gradient for the mine. The two graphs in Fig. 3.31 and 3.32 have been constructed for this purpose. The first graph displays the vertical stress against depth while the second plots the k-ratio or horizontal stress to vertical stress against depth.
Fig. 3.31 displays a linear relation between vertical stress and depth and is defined by:

$$\sigma_v = 0.88 + 0.032z$$

where σ_v = vertical stress in MPa
Z = depth in meters

Linear regression analysis suggests a goodness of fit equal to 97.6%. The linear relationship falls well under Herget's (7) (1982) extreme value gradient for the Canadian Shield and is close to the stress gradient proposed by Hoek and Brown (8) (1980) based on worldwide data. Also maximum and minimum values for horizontal stresses are included for every level. These were obtained from the estimated k-ratio gradient. As depth increases, the ratio decreases to unity from a maximum of 4.7 at the 400 level to unity at the 4000 level. Fig. 3.32 displays the average horizontal stress to vertical stress ratio with depth. Inconsistency occurs near surface on the 400 level while values become more consistent as depth increases. This is the usual trend observed as is displayed by the Hoek and Brown (8) (1980) variations according to world data. The curve was fit to the data points by visual inspection.

6.0 OBSERVATIONS AND ANALYSIS

6.1 Thompson Mine Wall Stabilities

The type of wall sloughing must be categorized for a proper wall stability evaluation. These categories are as follows: stable, which implies no sloughing; overbreak, which was attributed to the force of the explosives acting on the wall and is generally in the 0-0.9m (0-3 ft.) range; geological, which described the case where sloughing occurred to a clear geological contact of which the existence or whereabouts were unknown. The final category is simply slough where the instability of the wall rock or back produces unexpected ground failure.

Fig. 3.33 shows the histogram of the hangingwall slough. As described previously, roughly 30% of the database blocks experienced hangingwall slough. Also, the rock quality histogram showed very poor rock quality for some hangingwalls. At Thompson Mine the practice is to support the hangingwall with cable bolts only if there is an anticipation of hangingwall failure. To date, there has been few hangingwall failures in excess of 3m (10 ft.) and only 3 recorded in the database. Two extreme cases of 6.1m (20 ft.) and 7.6m (25 ft.) have occurred. Even if nature has provided the mine with weak walls enveloping the orebody, a relatively high dip angle (70 deg.) has minimized its costly impact. It is reasonable to assume that if the orebody had a lower dip in the 40 to 50 degree range hangingwall failures would be more predominant in the deeper portions of the mine. Fig. 3.34 shows a actual hangingwall and back failure at Thompson Mine.

A histogram of the back slough is displayed in Fig. 3.35. In the majority of cases no significant slough occurred. This is attributed to the competent nature of the rock when it is supported properly.

However, if the back has to cover a larger than designed span, which is usually created by hangingwall and/or footwall slough, then back sloughing will occur until a stable arch has formed. Extremely large back failures are rare. Only three cases of large back failures are reported in the database, one each with 12(40), 15(50) and 18m (60 ft.) of slough. Fig. 3.36 illustrates a large back failure at the Thompson Mine.

A footwall slough histogram is displayed in Fig. 3.37. Again there are very few failures. This is attributed to the stabilizing effect of the 70 degree dip.

6.2 Analysis of Wall Stabilities

In the following section correlations are made between the amount of slough and factors that are believed to influence stability. This is done in an attempt to determine the most significant factors of correlations influencing wall stability. The number of correlations has been limited to 15. Many more could have been included, but it was felt that the ones chosen represented best the ideas. Slough from hangingwall, back and footwall surfaces were investigated. Scatter plots are used to represent the data.

Each scatter plot is divided into between four and six groups in order to quantify trends in the data. The size of the groups were arbitrarily established for practical purposes. Each group of data contains a heading which displays the number of observations in the group and the percentage of sloughing incidences in the group.

For proper analysis of the following correlations, it is reminded that hangingwall slough occurred in 30% of all cases in the database while the back experienced an 11% occurrence of slough. Footwall sloughing amounted to 7% of all cases.

- **Hangingwall Slough vs. Length** (Fig. 3.38)

As expected the incidence of hangingwall slough is shown to increase with increasing block strike length. Sloughing occurred in 11.1% of the cases with strike lengths varying from 6 to 12m (20 to 40 ft.) The sloughing incidence increased to about one third of the cases in the groups with a strike length between 12 to 24.4m (40 and 80 ft.) Sloughing increased to 55.5% of cases in the 24.4 to 30.5m (80 to 100 ft.) group and then to 100% of cases in the 30.5 (100 ft.) plus range. The last group is not considered significant since it contains only one observation.

As the plot shows, the slough rarely exceeds 3m (10 ft.) in thickness. This thickness accounts for a large volume of dilution when taken over the area of the hangingwall.

- **Hangingwall Slough vs. Height** (Fig. 3.39)

It is expected that failure of the hangingwall is more likely to happen as the height of the block increases. The plot shows that only 4.5% of blocks experienced hangingwall slough with block heights of less than 27.4m (90 ft.) Block heights above 27.4m (90 ft.) do not show an increasing trend of hangingwall slough. With block heights between 27.4m and 64m (90 and 210 ft.) the incidence of hangingwall slough falls between 25.7% and 43.9%. No specific trend can be drawn from this correlation. However, in the 22 cases of the first group, that is less than 90 feet in height, only one case sloughed. Also, the only case above 210 feet in height sloughed.

- **Hangingwall Slough vs. Dip** (Fig. 3.40)

The gravity mode of failure is more apt to occur as the steepness of the dip diminishes. The group that has the highest percentage of slough is the 65 to 75 degree dip group with 39.7%. Included in this range is the average dip of the orebody for Thompson Mine. The second most common group is the 55 to 65 degree range where 29.4% of the cases sloughed. A full one third of the cases have sloughed in the 45 to 55 degree range. There is only one slough case reported where the dip is above 75 degrees. This correlation does not show the initial expected trend.

- **Hangingwall Slough vs. Shape Factor** (Fig. 3.41)

A long and narrow area will have a low shape factor while a squarer type surface will be characterized by a high shape factor. A low shape factor is indicative of minimum hangingwall slough and a high shape factor will be prone to a large amount of hangingwall slough.

The grouping of data clearly displays that as the shape factor increases, the percentages of cases of slough increased accordingly.

- **Hangingwall Slough vs. Quality** (Fig. 3.42)

Quality defined as the modified NGI rock quality index (Q) takes into account the block size factor and the shear strength of the joints that make up the individual blocks. The hangingwall quality measured values vary from 3 to just over 14, with the majority of the points situated in the 9 to 11 range. It is expected that the hangingwall slough should decrease in occurrence as the rock quality increases.

Except for one group which has only 2 cases, the expected correlation is obtained. When the rock quality is at its lowest range, the percentage of cases where sloughing occurred is 54.5%. As quality increases for the final two groups, the percentages decreased to 28.8% and 20% respectively.

- **Back Slough vs. Width** (Fig. 3.43)

There were 11 back failures in the 101 applicable cases in the database representing 11%. The initial expected trend from a plot of back slough versus width is that as the width increases the amount of slough should increase. This is not always the case. In longitudinal mining, an increased width will have the effect of increasing the back span leading to increased back slough. However, in transverse mining, an increased width is desirable as the strike length of the block is kept to a minimum. In the database, the ratio of longitudinal mining to transverse mining blocks is approximately 2:1.

The resulting analysis from grouping the data does however approximate the expected trend of increased back slough with increased width. There is a large increase in sloughing from the 12 to 18m (40 to 60 ft.) range. However, above 12m (40 ft.) widths, the amount of data diminishes quite rapidly minimizing the last three groups influence.

- **Back Slough vs. Length** (Fig. 3.44)

In longitudinal mining the strike length does not influence the back slough since the shape factor is kept to a minimum due to the narrow widths. This is evident from the longitudinal blocks in the grouping 80 to 100 feet which experienced no slough. In transverse blocks, which have greater widths, an increased length will initiate sloughing. Transverse lengths are usually kept small in relation to their widths. From the graph it becomes evident, the first group represents transverse back failures with the two following groups representing longitudinal back failures.

- **Back Slough vs. Shape Factor** (Fig. 3.45)

The graph displaying the occurrence of back slough versus the shape factor eliminates the uncertainty in distinguishing between transverse and longitudinal mining blocks because of the unbiased definition of the shape factor. The graph shows a positive correlation in that back slough increases as the shape factor increases.

- **Back Slough vs. Tons Removed** (Fig. 3.46)

The expected trend from this correlation is that the back slough will tend to increase as the amount of tons removed is increased. This can be attributed to the larger the block dimension thus a larger shape factor. This can be examplified by a large stope with a sizeable shape factor for the hangingwall. In this case, the hangingwall would more easily slough, therefore, increasing the width of the block and cause an increased shape factor for the back thus possibly resulting in back slough.

The graph displays the expected trend for blocks less than 18,140 tonnes (20,000 tons,) only 4.5% sloughed. For blocks between 18,140 and 36,290 tonnes (20,000 and 40,000 tons,) 8.3% sloughed. For blocks larger than 36,290 tonnes (40,000 tons) significant sloughing occurred in the back.

- **Back Slough vs. Quality** (Fig. 3.47)

The amount of back slough is expected to decrease as the rock quality increases. The rock mass quality values obtained range from about 6 to 11. The grouping of the data displays the negative correlation. Only 5% of the cases in the quality range of 9 to 11 have experienced sloughing with no cases above this range. Significantly more sloughing occurred below the quality value of 9.

- **Footwall Slough vs. Length** (Fig. 3.48)

In the database there are 9 footwall failures or slightly more than 7%. In examining the footwall slough and the strike length interrelationship, the trend expected is an increase in slough as the block length increases. This is displayed as the percentage of failures increases gradually from 3.6% to 11.1%. The last two groups contradict the trend but their low number of cases, only 3 in total, decreases their importance in the correlation.

- **Footwall Slough vs. Height** (Fig. 3.49)

A similar correlation to the previous figure is found in the plot of footwall slough against the height of the stope. The percentage of failures increases gradually from 0% to 5.7%. In the group 52 to 64m (170 to 210 ft.) height 20% of the footwalls sloughed. The last group has only one data point and its importance is minimized.

- **Footwall Slough vs. Dip** (Fig. 3.50)

The greatest chance of footwall sloughing occurs when the dip is vertical due to the toppling effect. The results show a relationship exactly contrary to the expected positive correlation. As the dip increases, the occurrence of sloughing diminishes indicating that other factors, such as, rock mass quality and blast damage are involved.

- **Footwall Slough vs. Shape Factor** (Fig. 3.51)

An increase in the shape factor should increase the occurrences of slough no matter which stope surface is being investigated. An anomaly is present again in the last group where no failures were reported. However, the first four groups show the expected trend.

- **Footwall Slough vs. Rock Mass Quality** (Fig. 3.52)

As rock mass quality improves, a reduction in the occurrence of footwall slough is expected. This relationship is well displayed in the plot. There is a 9.1% occurrence of slough in the 3 to 6 quality range group and this occurrence diminishes to 0% in the 12 to 15 range group. The 6 to 9 range group had one failure in three cases. The resulting percentage is high for this group, but its relative influence is reduced because of only 3 cases.

6.3 Cost of Instability

The graph in figure 3.53 displays the cost of dilution for a typical 300 ton per day stope. The potential revenue is plotted against the feet of hangingwall slough. The graph portrays a rough estimate of the potential revenue loss. As demonstrated earlier, most hangingwall slough rarely exceeds 3m (10 ft.)

If 1.5m (5 ft.) of hangingwall sloughs, this would in effect reduce the potential revenue to 95% of its expected level. Similarly, 3m (10 ft.) of hangingwall slough would only allow 90% of the potential revenue. The savings potential cannot be overstated as it represents large economic improvements.

6.4 Hangingwall Behaviour

As mining increases at deeper elevations of the mine, the virgin stress levels increase accordingly. This adversly affects the hangingwall behavior. Based on numerical modelling results from a similar bedded orebody by Meek and Beer (11) (1982) values of up to 75 mm are not uncommon for hangingwall displacements as shown in figure 3.54. The rate of displacement actually decreases, because of virgin stress change, after reaching a depth of 500 meters.

The influence of the foliation spacing is another important factor in hangingwall behavior. Foliation is present in the weak biotite schist walls at Thompson Mine. As the spacing of these bedding planes diminishes, the displacements increase. On figure 3.55, again based on the numerical modelling just mentioned above, a spacing of 6 meters reflects in a 50 mm displacement while a spacing of 2 meters would indicate a displacement of 75 mm. Figure 3.55 also displays that the maximum amount of slip actually decreases with increasing bedding plane frequency.

Hangingwall behaviour is influenced by the length of the span. Increased movement or displacement of the wall plates increases as the span increases, displayed in figure 3.56.

The development of a hangingwall failure hypothesis is presented in figure 3.57 by the three stages of hangingwall failure. If an excavation is made in pre-stressed rock with the major principal stress perpendicular to the hangingwall, the hangingwall and the footwall will deform due to the release of stress at the free surface. The result will be that the rock material becomes completely destressed in the direction perpendicular to the hangingwall except at the abutments where a pressure arch will form. The destressed material will tend to open up cracks or bedding planes. At the abutments, high shear stresses are present and will cause slip or movement on the bedding plane. The compounding effect of the shear stresses at the abutments and the tensile stresses perpendicular to the bedding will initiate the formation and movement of individual rock plates, as well as, opening of cross fractures.

It is these plates that are either stable or unstable and their stability is dependant on the following factors; the thickness of the rock plate, the frequency of cross-fractures, the presence and location of ground support, and the stope dimensions. If instability occurs, equilibrium will not be attained until the hangingwall plates attain a certain thickness or there is a definite decrease in span. The modelling indeed is confirmed by what has been seen at Thompson Mine such as the slabbing from the hangingwall. Again, the behaviour noticed at Thomson mine confirm the modelling performed by Meek and Beer (11) (1982).

6.5 Back Behaviour

At Thompson Mine, two potential types of back failures have been observed: the wedge type failure and the arching type failure. The two are displayed in figure 3.58. The occurrence of wedge type failures is rare because the orientation of the joint sets are not conducive to this type of failure. An arching type failure is the more likely back failure mode. In the majority of cases at Thompson Mine, the arching type failure is initiated by either a major hangingwall or footwall failure. The ground support system used was in effect not designed for the increased span of the back. Equilibrium is achieved once a stable arch has formed. The height of this arch is variable and remains an intriguing design question with respect to ground support.

As previously mentioned, the stress levels increase as the depth of excavations increases. At shallow depth, stability problems are usually structurally controlled and involve gravity falls of blocks from the back and sidewalls. These are the wedge type failures. At greater depths, failure becomes stress induced. At Thompson Mine there is also an increase in shear stress at the abutments resulting in an arching type failure. The problem lies in the meshing of structural and stress related failures at a certain depth. There exists a large grey zone where both types of failures can manifest themselves.

A scatter plot of Back Slough versus Depth is presented in figure 3.59. The back failures range from 305 to 701 m (1000 to 2300 ft) in depth. The plot does not show the expected increase in back slough with depth. This is in part due to the narrower ore zone widths in the lower levels of the mine and in part due to higher levels of extraction (therefore higher induced stress levels) in the upper part of the mine.

7.0 SELECTION OF STOPE DESIGN METHOD

7.1 Precedent

The experience gained at Thompson Mine can best be displayed by the scatter plots shown in figures 3.60 and 3.61. An initial attempt was made to to correlate the size of open stope surfaces, that is the hangingwall and back surfaces, to their relative stability. In figure 3.60, stable and unstable cases are plotted for the block heights versus the block lengths. An arbitrary curve has been applied where most of the points below the line were stable while most of the points above the line have experienced failure. This hangingwall failure line can serve as a rough design guideline for VBM blocks, as it gives an indication of feasible stope dimensions.

Figure 3.61 represents a similar concept, only it is applied to the back. Stable and unstable points are plotted for the length against the width of individual blocks. A similar curved line is applied where all points lying below the line can be expected to be stable points above the line have a greater chance of experiencing instability. It must be emphasized that the two plots shown relate past experience with respect to the hangingwall and back stabilities and serve only as rough guidelines for designing stable VBM blocks.

7.2 Analytical/Numerical

Numerical models are used in order to aid in the engineering design of underground excavations. They permit the engineer to predict the magnitude and orientations of induced stress due to the redistribution of virgin stress caused by the process of mining.

There are many models available today, each having different capabilities and limitations. Some models can only be used for two dimensional problems, assuming plane stress or plane strain conditions while others can be used to model three dimensional problems.

Based on how the rock mass behaviour is modelled, either a continuum approach or discontinuum approach is used. In the continuum approach, the rock mass is assumed to be a continuous medium with few or no significant geological discontinuities. This approach, being easier to model mathematically is the most developed and by far the most used. In addition,by assigning various rock mass properties to the medium, differing rock mass behaviour may be modelled. In hard rock mining, linear elastic and non-linear elastic media are usually assumed. The discontinuum approach assumes that the rock mass is an assemblage of blocks which are capable of sliding and rotating. This approach is much more difficult to model and is still in the early stages of development.

Continuum approach models can be seperated by the mathematical method used to solve the problem. Differential methods approximate the solution for the entire domain while integral methods approximate the solution for the excavation boundary. Differential methods are known as the finite element method (F.E.M.) and integral methods are commonly called the boundary element method (B.E.M.).

The finite element method seperates the entire medium except the excavation by quadilateral or triangular elements as shown in figure 3.62. Principal stress magnitudes and orientations are displayed at the centroid of each element and displacement vectors are located at the nodal coordinates. Scale factors of 1 and 50 are used for the stress magnitudes and the displacements respectively. This example (figure 3.62) comes from a linear elastic stress analysis study done for the 1-C orebody at Thompson Mine using the QUAD finite element program (18)(1990).

Since boundary element methods approximate the solution at the excavation boundary only, computation time is reduced significantly when compared to the finite element method. For this reason, the boundary element method is an excellent candidate for more complex three dimensional problems. The B.E.M. assumes that the medium is infinite or semi-infinite. An example is the Inco Limited (Ontario Division) BEM 2.2 program. It was used to investigate the stress concentration that would be produced in the 1803-331 crown pillar by excavating the lower portion. Stresses in excess of 68.9 MPA (10,000 psi) were estimated. The results are shown in figure 3.63. Another program, M-BEM which is the mainframe version of BEAP (19)(1990) was used to calibrate the model with the results from the monitoring of the drift located near the excavation in figure 3.64. Principal major and minor stresses are included at various points surrounding the stope boundary along with the displacements vectors.

MINTAB (20)(1982) is a pseudo three dimensional displacement discontinuity model based on the integral method. Originating from South Africa to model reef type deposits, the tabular oreboby is modelled two dimensionally by square elements with the third dimension being a uniform thickness. The host rock surrounds the orebody to complete the seam. Displacements are calculated at unmined and backfilled points in the seam. Figure 3.65 displays the mining step number 6 or the removal of the first block in the 1803-331 stope. Increased stresses perpendicular to the orebody are seen immediately to the left of the excavation as well as above and below. The area to the right of the excavation is a mined out portion which was subsequently backfilled. The letter A means that the backfill carries an insignificant amount of stress while the higher letters are carrying the most stress.

7.3 Empirical

The methods available to a mining engineer in order to empirically assess stope stability are the Mathews' method, Potvin's modification and Laubscher's method. All these variations of empirical analyses have their results plotted on graphs which are displayed in figures 3.66, 3.67 and 3.68 respectively. The methodologies are explained earlier in section 3.3.1. Information regarding the detailed calculation is included in appendix 3B.

The Mathews' stability graph in figure 3.66 displays three zones; stable, potentially stable and potentially caving. Two grey areas are used to seperate the three zones.

Potvin's modified stability graph, shown in figure 3.67 has two zones, stable and caved, seperated by a grey area. Potvin's graph displays less vagueness than Mathews' and has the empirical advantage of drawing from a larger database.

The graph in figure 3.68 displays Laubscher's relationship. By plotting the adjusted rock mass rating versus the hydraulic radius, one would obtain a stable exposure if the plotted point was located on the left side of the far left curve. The second curve gives an estimate of the limiting point where dilution becomes excessive and is finally termed caving.

8.0 APPLICATION OF THE STOPE DESIGN METHOD

The Thompson Mine database contains data on 126 individual mined blocks. The Thompson Mine longitudinal showing trial V.B.M. stopes is shown in figure 3.69. A series of six stability graphs will be described in the following pages which makes the maximum possible use of the database. The hangingwall, back and footwall surfaces were analysed using the stability graphs from both the Mathews' and Potvin's empirical models.

Two important points to note before describing the stability graphs. Firstly, most points plotted were grouped close together, therefore, in order to help the visual examination aspect, only the portion of the graph in which the points are concentrated will be displayed. Secondly, in the case where two points plot over or overlap for more than 50% of its surface area, only one point was plotted for purposes of clarity. Therefore, a visual count of the points may not necessarily add up to the total number of cases displayed on the title of each graph.

8.1 Hangingwall Experience

The results of the hangingwall surface stabilities are as follows; 85 stable surfaces, 37 unstable surfaces. Four cases had to be eliminated due to incomplete data or the analysis was not considered applicable.

The stability chart for the hangingwall experience using the Mathews' stability graph is shown in figure 3.70. The first trend that is drawn from this graph is that all the points are clustered in one small area of the graph. This phenomenon can be attributed in part due to the prediction of tensile stresses from numerical models. This had the effect of defaulting the stress factor C to a value of 1.0. Secondly, all the unstable points plot in the potentially unstable grey zone or down and to the right of this zone. There are no unstable points in the stable zone. Thirdly, the plot of the stable points are scattered throughout the stable and unstable portions of the graph but no points plot in the caving portion of the graph.

The conclusion that can be drawn from this empirical analysis is that the Mathews' graph displays some validity and can be used as a rough design guideline to predict hangingwall failure.

The Potvin modified stability graph showing the hangingwall experience is displayed in figure 3.71. This graph contains only one grey area which is defined as unstable. In comparing Potvin's graph with Mathews' graph, one notices that while they appear to be very similar, the actual coordinates of the points differ. This is due to the modifications done by Potvin in the calculation of the stability number N. Again all of the points are clustered due to the prediction of tensile stresses in the hangingwall which has the effect of defaulting the stress factor A to the value of 1.0. Potvin's graph shows that over 95% of the points plot in the stable and unstable zones, the majority of those being in the stable zone. The unstable points do not correlate very well with the modified failure curve as many plot in the stable zone. However, the stable points correlate moderately well with the failure curve as only three points plot in the the failed portion of the graph.

The use of the modified stability graph is not suggested since the graph is beyond an acceptable correlation range. Further research will be undertaken to help determine how a variety of factors, such as blast damage for example,influences the results of the graph.

8.2 Back Experience

The next stope surface examined is the back. There were 90 stable surfaces, 11 unstable surfaces and 25 cases of incomplete data or the analysis was not considered applicable. Investigation of the Mathews' stability graph reveals that due to relatively low calculated values of the factors A and C for the majority of stope backs, the stability number N results are equally very low. The stability number was so low that all of the points, or 101 applicable cases, plot in the potentially unstable and caved portions of the graph as displayed in figure 3.72. Since this does not correlate with field experiences it can be concluded that the Mathews' analysis of the Thompson Mine backs is not applicable.

However, with Potvin's modification of having only one grey area and in the manner in which the stability number is calculated, the results shown in figure 3.73 for the backs are quite similar to those obtained for the footwall experience.Most stable points plot in the stable zone but 13 points plot in the caved zone. In referrence to the unstable points, only 1 point plots in the stable zone while 7 are situated in the unstable zone and 3 in the caved zone.

In retrospect, it is difficult to choose which graph one should use because all backs at Thompson Mine are conditioned with a variety of ground support mechanisms. The consequence of this ground support is a varying stabilizing effect which cannot be accounted for in either empirical model.

8.3 Footwall Experience

The final stope surface studied is the footwall. Thompson Mine enjoys a relatively favourable dip averaging 70 degrees for the footwalls. Nonetheless, nine failures were recorded out of 124 cases. There were 115 stable surfaces and 2 cases of incomplete data or the analysis was not considered applicable. The Mathews' graph displaying the Thompson footwall experience is shown in figure 3.74. Similar results were obtained when comparing this graph to the previous Mathews' hangingwall experience. Acceptable correlation is obtained with the stable points, as they all plot in the stable or potentially unstable areas of the graph except for one point which is situated in the caved area. All of the points which have experienced failure, plot in the potentially unstable area of the graph. Again, all the points are clustered in one area of the original graph.

In examining the footwall points on the modified stability graph, which is displayed in figure 3.75, the stable points exhibit a greater tendancy to be situated in the stable zone rather than in the potentially unstable zone of the Mathews' graph. However, 11 stable points plot in the caved zone. Potvin's graph seems to be well calibrated for Thompson Mine footwalls as the unstable points plot in the unstable and caved zones. There is one point questionable as it lies on the border between the stable and unstable zones.

In conclusion, both graphs could be used as rough design guideline for predicting footwall failure with preference being given to the modified stability graph as the data is much better distributed in their respective zones and calibration appears adequate.

9.0 PILLAR DESIGN

9.1 Introduction

Once the important stage of stope stability has been addressed, it is of paramount importance to integrate these considerations into the broader view of maintaining the global stability of the mine structure. The two seperate design issues must gel into one.

Brady and Brown (3) (1985) suggest that in any economic mine design, the amount of ore committed to pillar support should be kept to a strict minimum while maintaining global mine stability. There is therefore a need to understand the properties and performance of pillars and pillar systems to attain the maximum economic potential of an orebody in a safe manner. Controlling the rock mass displacements throughout the zone of influence of mining using pillars as support is termed a naturally supported mining method.

Vertical block mining is a naturally supported mining method. The stability acheived results from the generation of ore remnants or areas of sub-economic grade acting as pillars. If the field stresses are high relative to the rock mass strength, then a substantial proportion of the proven mineral reserves must be committed to pillar support. In metal mines however, reserves are limited and the present trend is towards 100% extraction and minimum pillars. The life of a mine can be directly related to the economic recovery of a high proportion of pillar ore. Pillar recovery cannot commence until the open stopes have been filled. Backfill is used as a structural component and helps to reduce the possibility of wall rock or crown pillar collapse. It is also effective in restricting large displacements of the host rock.

The concept of pillar design can best be explained by visualizing a room-and-pillar operation. Due to stoping activity, the pre-mining stress will be redistributed and cause an increase in pillar loading as shown in figure 3.76. If states of increased stress levels are less than the in-situ rock mass, the pillar will behave in a linearly elastic manner. Rupture in the body of the pillar will occur when states of stress have attained its peak load-bearing capacity. Failure will either be carried out gradually or violently as in the case of a pillar burst. If a gradual failure is observed, the pillar will still retain some load bearing capacity, although at a lower level. This is termed the residual strength and it can amount to roughly 30% to 50% of the peak pillar strength where the pillar is contained by surrounding backfill.

There are three principal modes of deformation behaviour for mine pillars that have been observed and they are displayed in figure 3.77. In the first case, the pillar rupture mode involves spalling from the pillar surfaces to produce a pillar which resembles an hourglass. They are usually seen in relatively massive rock where pillars have a low height to width ratio. Secondly, pillars which have a high height to width ratio usually fail from the creation of inclined shear fractures as seen in laboratory uniaxial compressive strength tests for coarse grain samples. The third major mode of behaviour involves the presence of highly deformable walls which form the interface between the pillar and the host rock walls. In this case, transverse tractions over the pillar surface promotes internal axial splitting which can be visually observed by the bulging of the pillar surface.

9.2 Strength

In carrying out pillar design, it is important to predict the in-situ performance of a pillar. This can be obtained by a method which assesses the ultimate strength or peak resistance of the pillar to the axial compression.

The first area of instability in a pillar is at its mid-height centerline. This is where the normal stress is at its highest and where there is the lowest confining stress. In pillar design, the direction of greatest importance is usually the direction that has the highest stress. For inclined orebodies such as Thompson Mine, the largest induced load in the pillar is perpendicular to the orebody.

An empirical relationship has been derived by Hedley and Grant (6) (1972), based on the retrospective analyses, which suggest that the strength of a pillar is related to its geometric shape and actual dimensions. It is expressed in the form;

$$Q_u = kW^a/H^b$$

where:

Q_u = the pillar strength (psi),
k = the strength of 1 cubic foot (UCS12) of pillar material,
W = pillar width
H = pillar height
^{a} = 0.5
^{b} = 0.75

To obtain UCS12 many authors suggest a scaling factor for the compressive strength of a 2 inch diameter specimen (UCS2) where;

$$UCS12 = 0.7 * UCS2$$

Hedley's formula is the only one available for hard rock mining and is widely used for open stope design. It is based on data from hard rock room and pillar mining at Elliot Lake. The database consists of 23 stable pillars, 2 partially failed pillars and 3 crushed pillars. Hedley suggests that a safety factor of 1.5 or larger will yield a stable pillar while a safety factor near 1.0 would be crushed. The following table diplays the calculated pillar strength obtained for various width to height pillar ratios.

Hedley's Formula

Width/Height	Q_u (MPa)
0.5	10.9
1.0	18.4
2.0	21.9
3.0	26.8
4.0	31.0

Another method which can be used to estimate the average pillar strength is the Hoek and Brown criterion which is displayed in figure 3.78. The series of curves are based on numerical modelling and the distribution of failed rock inside pillars of different shapes. Each curve can be considered a pillar failure criterion for a specific rock mass quality. The m and s constants account for pillar volume and structural defects. For Thompson Mine, the rock mass can be considered of good quality with m and s values of 1.7 and 0.004 respectively. The uniaxial compressive strength of intact rock samples is 70 MPa. A quick calculation of width to height ratios yields the following data.

Hoek and Brown Failure Criterion

Width/Height	σ_{IS} (MPa)
0.5	10.5
1.0	21.0
2.0	42.0
3.0	56.0
4.0	70.0

where:

σ_{IS} = Average Pillar Strength.

In comparing both series of results, it can be seen that Hedley's formula is conservative relative to the Hoek and Brown criterion. However, his formula was designed for small pillars in a room and pillar environment and is consequently much less conservative in that situation. The Hoek and Brown strength curves were practically applied by Potvin (15) (1985) and Page and Brennan (12) (1982) for the good and fair rock mass qualities with successful results.

In order to assess the stability of pillars, an empirically developed relationship was suggested by Hudyma (15) (1988) and is called the pillar stability graph (figure 3.79). The x-axis plots the pillar width to height ratios and the y-axis plots the normalized load on the pillar. The normalized load is defined as the estimated pillar load divided by the the uniaxial compressive strength of the rock. The pillar stability graph does not include correction factors for volume and rock mass quality. Hudyma suggests that there is not a great degree of precision inherent to the graph. However, the cases studied did display a definite trend. The failure area to the left and the stable area to the right are seperated by a transition zone. The bottom line suggests the start of mining problems due to pillar instability while the top line defines a criterion where pillar failure has been observed. In order to complete the graph, pillar load approximations are needed. They were obtained by numerical modelling and this topic is covered in the following section.

In examining the graph, it can be mentioned that all the data was obtained from Canadian open stope mines. Also, there are very few stable pillars with a normalized load greater than 0.33 and no stable pillars have a normalized load above 0.5. The pillar stability graph can be used as an effective guide to design pillars. This method has the advantage of being able to apply to open stoping operations. Hedley's formula can be shown to be conservative for higher width to height ratios and the Hoek and Brown good rock mass quality strength curve (applicable at Thompson Mine) plots for the most part inside the transition zone.

9.3 Load and Sequencing

As discussed previously, in order to help predict the state of pillar stability, one must be able to estimate the actual amount of loading on the pillar. There are two methods which can be used. The first is the tributary area analysis and the second is by numerical modelling techniques.

The tributary area analysis assumes equal and symmetric stress redistribution once stoping has been performed. It is a method which ignores numerous factors in its simplicity. It can easily be shown that the predicted loads obtained by this method are nearly always over estimated. Hudyma (15) (1988) suggests that the over estimation can easily attain 100%. Nevertheless, the tributary area analysis provides a quick and easy solution for estimating pillar load. By using this method, one can plot the stress concentration factor of the pillar against the area extraction ratio as seen in figure 3.80. If a mine uses only natural pillars for support, the extraction ratio would rarely surpass the 75 to 80% range since there would be extremely large incremental changes in the stress concentration factor from only a small change in the extraction ratio.

The second method to estimate pillar loading is by making use of numerical models. It is a more scientific approach and yields output of much greater precision than that obtained by the tributary area analysis.

Models can be easily set up to predict the stress behaviour of the backs in both the topsill and the bottomsill for example. Particularly useful are the obtainment of the stress concentartion factors when compared to in-situ pre-mining stress conditions. However, one must keep in mind the present trend of 100% extraction such as longitudinal mining which does not leave any pillars to attract stress but rather uses block backfilling to restrict host rock movement and help, to some extent, carry stress.

It is therefore of greater practical value to study the effects that various mining sequences have on regional pillar loading. At Thompson Mine, the utilization of numerical models to investigate the effects of various mining sequences can best be shown by the three MINTAB runs completed for the future mining of the 1-C orebody.

The 1-C orebody consists of a relatively wide and tabular orebody averaging 24 to 30 m (80 to 100 ft) in thickness with a strike length of about 305 m (1000 ft). The orebody is contained within the 2400 and 3150 level with an average dip of 45 degrees. Transverse open stoping is the projected mining method. The open stope dimensions modelled are usually 18 m (60 ft) on strike with a height along dip of of 30.5 m (100 ft). Each element (letter or number) displayed in the following figures have dimensions of 6.1 m (20 ft) wide by 6.1 m (20 ft) high.

Three different sequences were examined in this study. All of them consider 100% extraction. The first extraction sequence considered the start of mining from the bottom and retreating upwards to the 2400 level in one mining front, as shown in figure 3.81. The number zero corresponds to unmined areas, the number 1 displays a previously mined area and the number 2 represents part of the orebody which was defined as early ore. The first mining step of interest is step number three at the bottom. Then each consecutive block is mined according to its sequential number. The letters of the alphabet are used after the mining step number 9.

The second sequence considered was a primary-secondary method where pillars were recovered relatively quickly in order not to leave any ore behind. It can be viewed as a primary mining advance with a close secondary pillar recovery. This sequence is displayed in figure 3.82.

The third sequence can be termed a true primary secondary mining method. In this case, all the primary blocks are mined with a first pass and then pillar recovery is undertaken. Figure 3.83 shows this mining sequence.

The results from the sequencing analysis shown in figures 3.84, 3.85 and 3.86 suggest the following. With respect to the first sequence or the single mining front, it can be said that while the increased stresses due to mining on the bottom and right abutments are of no concern, the stress wave created on the left abutment and the back are of great concern. This would make mining conditions particularly difficult. Stresses reach a critical level (40 MPa) in mining step 13 and as mining proceeds past this point, the surrounding stresses increase incrementally. Therefore every mining step beyond step # 13 will experience highly stressed backs and left abutments. Conditions gradually worsen in the later stages of mining.

In the second approach, that is primary extraction with closely following secondary extraction, we can see that in the early mining steps, there is the gradual loading of the secondary pillars. In step number 7, the first pillar is removed. This has the effect of transferring its load to the abutment on the right and the closest pillar to the left. As mining progresses to step number 12, two pillars of 91.5 m (300 ft) in height are being loaded to capacity. They can be designed to fail and mined at a later time. As mining approaches the top, the crown pillar takes progressively more load. With this sequence, primary mining occurs in relatively low stress areas, that is virgin mining conditions, and failed pillars can then be mined with minor operational problems. One question to be investigated is on the manner in which these pillars will fail. Will the failure be gradual, or violent as in the case of a rockburst. Also a serious limitation is inherent in the program with regards to the rock mass behaviour. The program assumes that the rock mass behaves in a linear elastic manner. Therefore, the rock is unable to fail and consequently redistribute its load to the surrounding areas.

The main advantage of this sequence is that mining is performed in relatively unaffected areas. Secondly, pillar recovery commences as soon as the pillar has reached the post peak phase. As a result, pillar mining can be performed in residual stress conditions where there is less a chance of operational problems.

The third option also consists of a primary secondary sequence only with the secondary extraction not beginning until the primary pass is completed.

This option creates very long rib pillars of 18 m (60 ft) in width seperated by 18 m (60 ft) wide mining blocks. The load perpendicular to the orebody is then spread out evenly and it is not until mining step number 14 that we see a hint of high stresses in the middle of one rib pillar. Therefore, a 50% recovery of the orebody is possible without any anticipated ground problems as shown in mining step number 16. However, as soon as the first pillar is mined to the far right, a good proportion of the other pillars become overstressed. This is due to the fact that the pillars were loaded close to their peak load bearing capacity. By the simple removal of one structural member, all others become overloaded, although progressively less as the distance from the removed pillar increases. If these rib pillars can be designed to systematically fail, this option could prove to be quite efficient.

10.0 CONCLUSIONS

Thompson Mine is in the final conversion stages from cut and fill mining to VBM mining. Of the open stopes, about 65% are mined longitudinally while the balance is mined transversely. The typical Thompson Mine open stope dimensions are small in comparison to the typical Canadian open stope dimensions. However, Thompson Mine is characterized by a weak schist biotite hangingwall. This indeed limits the size of the open stopes.

The most severe ground control problems at Thompson Mine are focused on dilution, either of sandfill and/or rock, and blast damage. These problems should be addressed accordingly. Hangingwall and back failures have a serious impact on the amount of dilution and create various mining difficulties.

Prediction models take into account the rock mass characteristics, the effect of stress redistribution and stope spans to estimate the relative stability of an open stope surface. The Mathews' method and its derivative, the Potvin modification are the two empirical prediction models available to the stope design engineer today. Effort should be made to blend this model with past experience. The flow sheet process describes the logical path one takes to design stable open stope surfaces. Numerical models are used as tools to give an indication of the induced stresses due to the creation of an opening. The back analysis technique allows for model calibration for a particular mine site or mining domain.

Rock mass quality values for Thompson Mine oscillate near the value of 10. Values of 10 and above comprise the feasible range of open stope mining. RQD's are situated in the 85-90% range. Intact rock strength for the ore and the schistose walls hover around the 70 MPa value. A vertical stress gradient of Sigma v = 0.83 + 0.032z has been defined for the mine site. The linear relationship falls well under extreme Canadian Shield stress gradient.

The Thompson Mine wall stabilities have been investigated. Of all the stopes surfaces studied, hangingwall slough occured in 30% of the cases, back slough in 11% of the cases and footwall slough in 7% of the cases. Fifteen correlations relating to wall stability were analysed. All the data was grouped in order to make apparent the trends present. Most plots showed the expected trend rather well.

The effect of dilution on the potential revenue showed a significant savings potential.

The possibility of hangingwall failure increases with depth, length of span and the decrease in foliation spacing.

Back behaviour at Thompson Mine has only shown the arching type of failure when there is instability.

In selecting a stope design method, one could solely base his judgement by relying on past experience such as a simple graph displaying stable and unstable cases for the hangingwall surface. However, today more can be done. Numerical techniques can be used to obtain the the magnitude and orientations of induced stress caused by the process of mining. A series of numerical models, each having different capabilities and limitations are available. Former mainframe programs are now PC based. Also, empirical techniques are available to assess stope stability. The two techniques, Mathews' and Potvin's models, are used in conjunction with one another to help design stable stope surfaces.

The stope surfaces from the Thompson Mine database were analysed using the stability graphs from both the Mathews' and Potvin's model. With respect to the hangingwall, the Mathews' graph displays some validity and can be used as a rough design guideline to predict hangingwall failure. The modified stability graph is not suggested at present since the graph is beyond an acceptable correlation range.

The Mathews' graph for the back experience is not applicable due to the very low stability numbers obtained. In this case, all points plotted in the caved or potentially unstable portions of the graph. Acceptable correlation is obtained with Potvin's modification but it is difficult to account for the stabilizing effect of the ground support.

The graphs displaying the footwall analysis can both be used as rough design guidelines for predicting footwall failure with preference being given to the modified stability graph.

Integration of the stope stability considerations to the broader view of global mine stability is essential. Strengths of pillars which assure global stability can be assessed by empirical relationships such as Hedley's formula and the Hoek and Brown criterion. The actual load on the pillars can be estimated by either using the simplistic tributary area analysis or by numerical modelling techniques. Examples of three mining sequences for the 1-C orebody showed the expected gradual loading of the rib pillars, abutments and the crown pillar.

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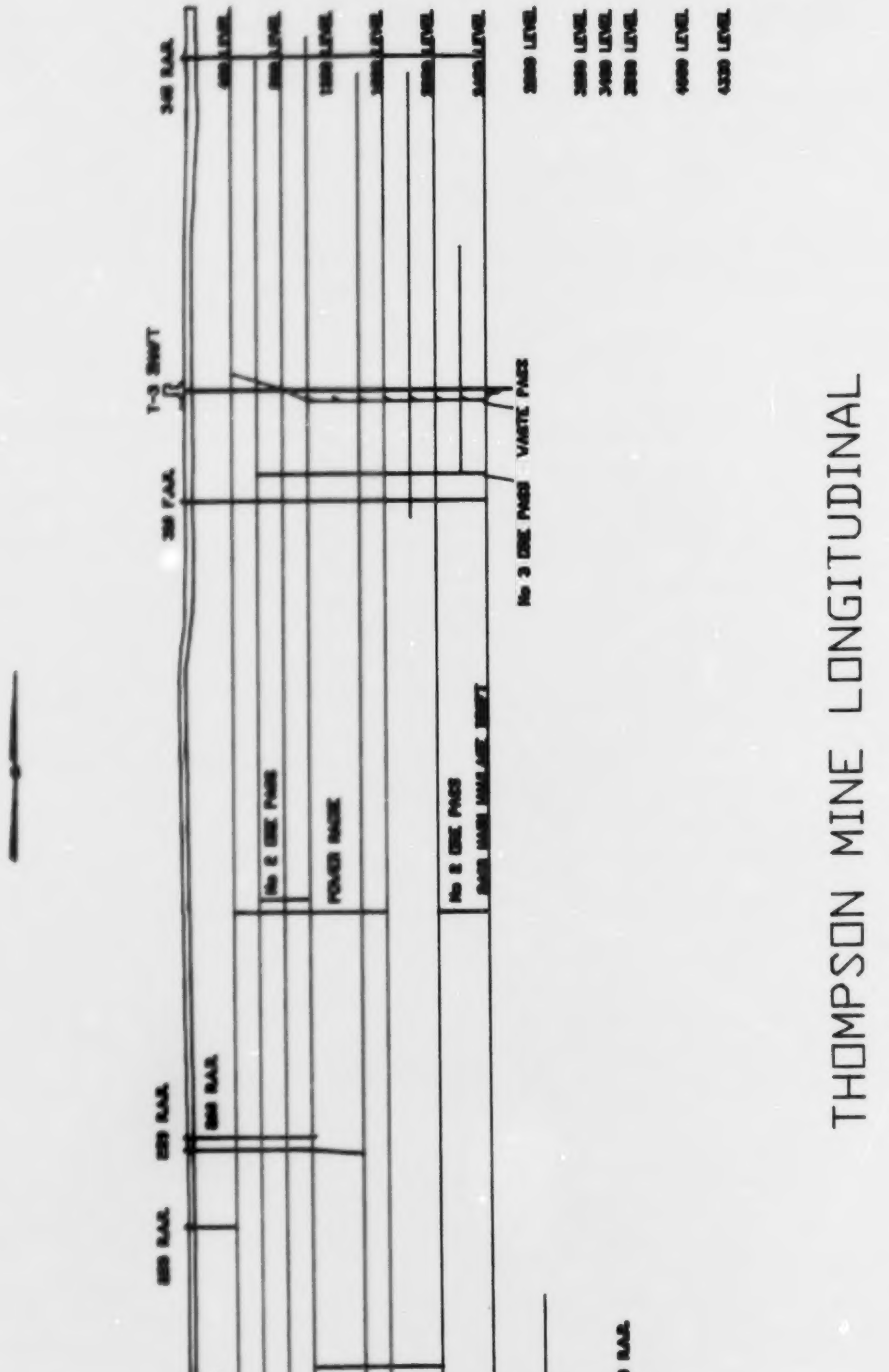
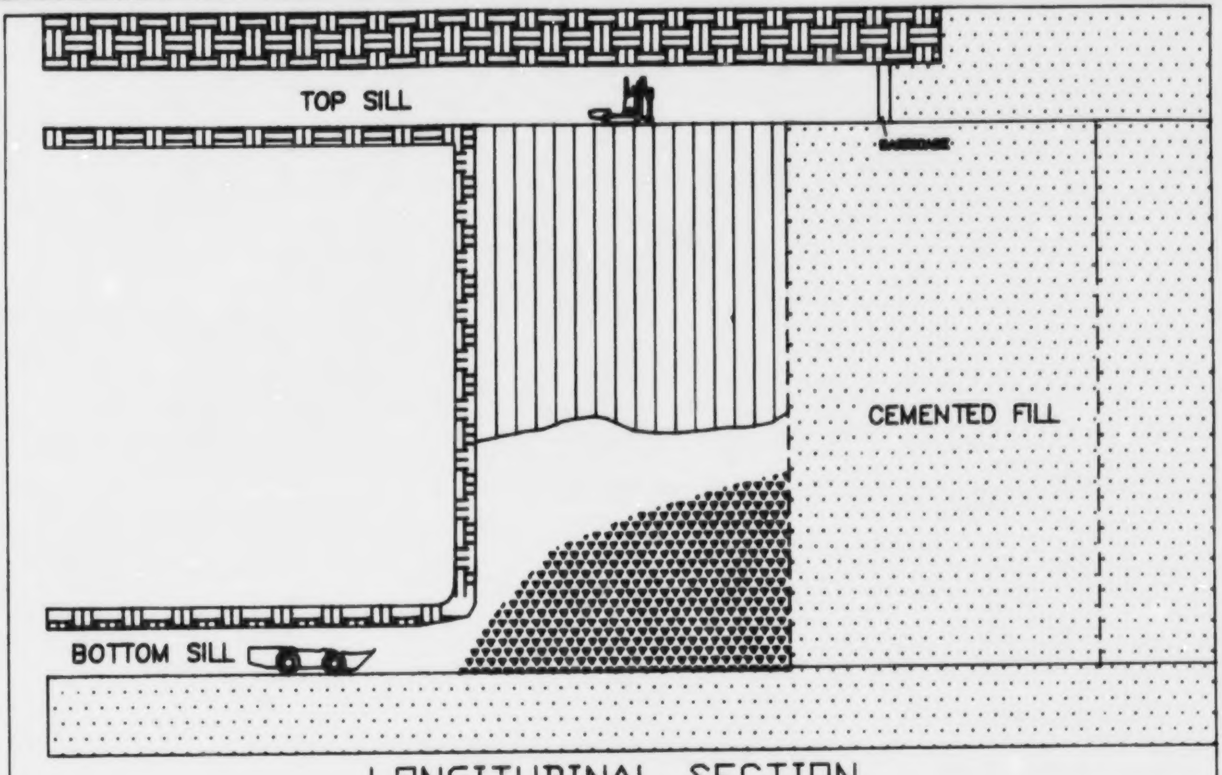
THOMPSON MINE LONGITUDINAL

FIG. 3.1

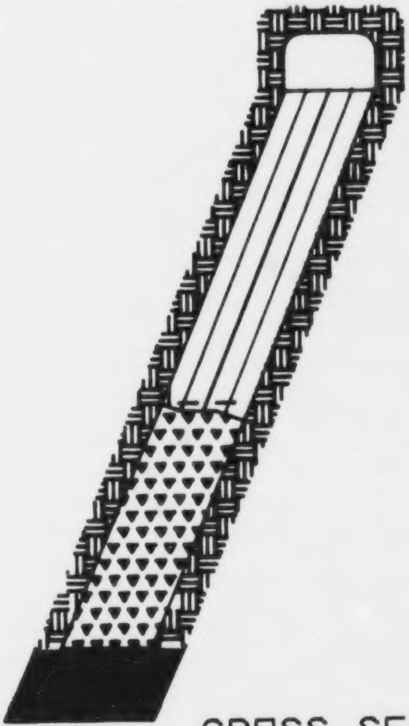

T-3 SHAFT
WASTE PASS
No 3 ORE PASS
POWER HOUSE
T-1 SHAFT
WASTE PASS
THOMPSON MINE LONGITUDINAL

LONGITUDINAL MINING

FIG. 3.2


TOP SILL
CEMENTED FILL
BOTTOM SILL

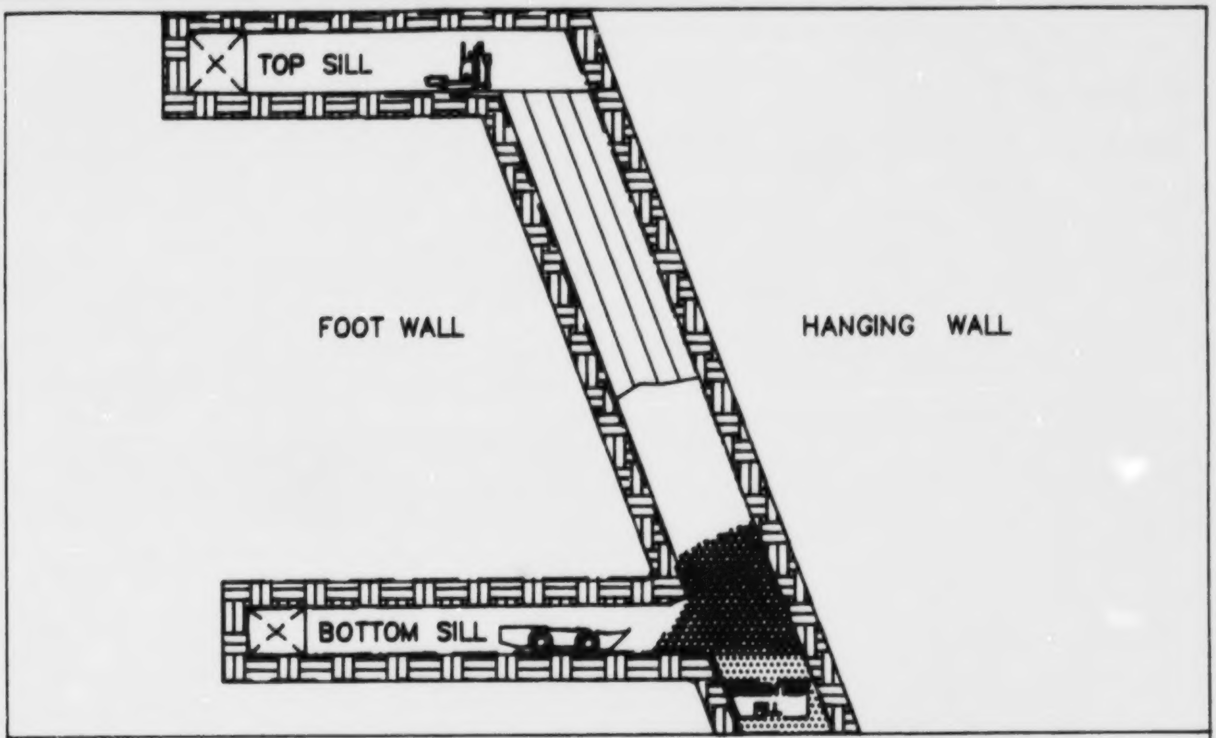
LONGITUDINAL SECTION



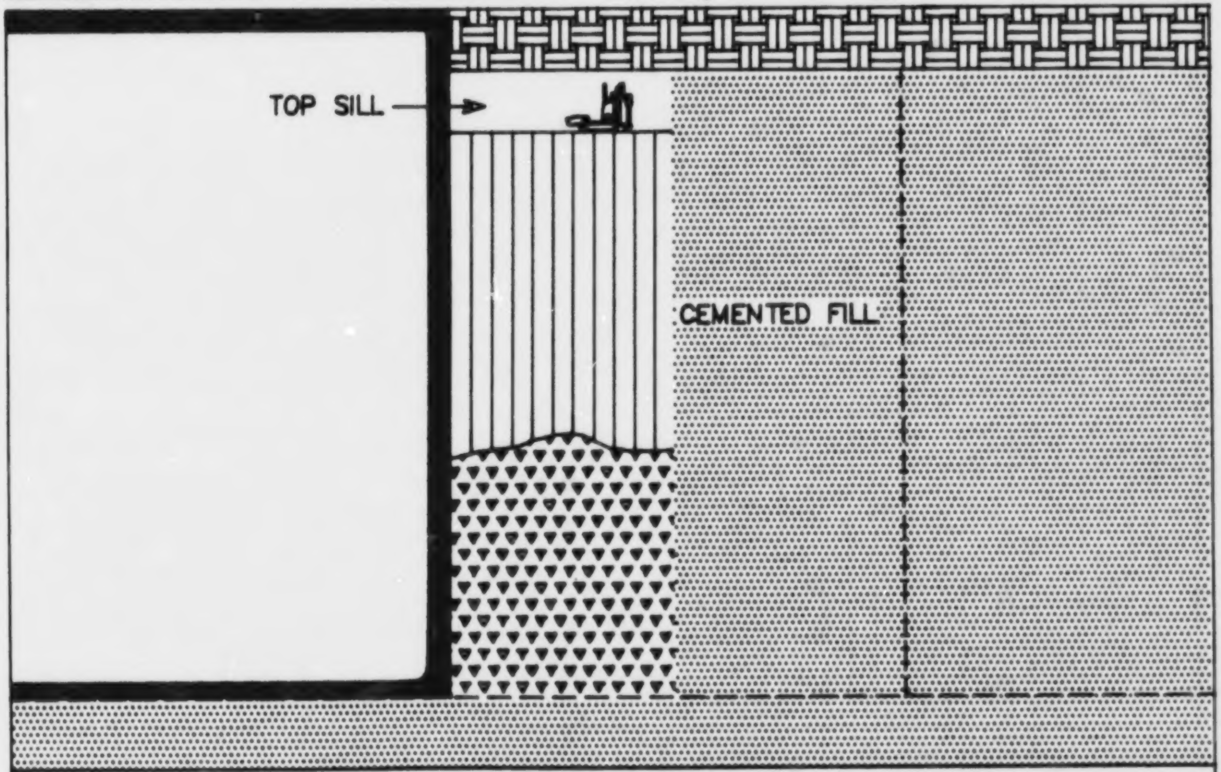
CROSS SECTION

TRANSVERSE MINING

FIG. 3.3


TOP SILL
FOOT WALL
HANGING WALL
BOTTOM SILL

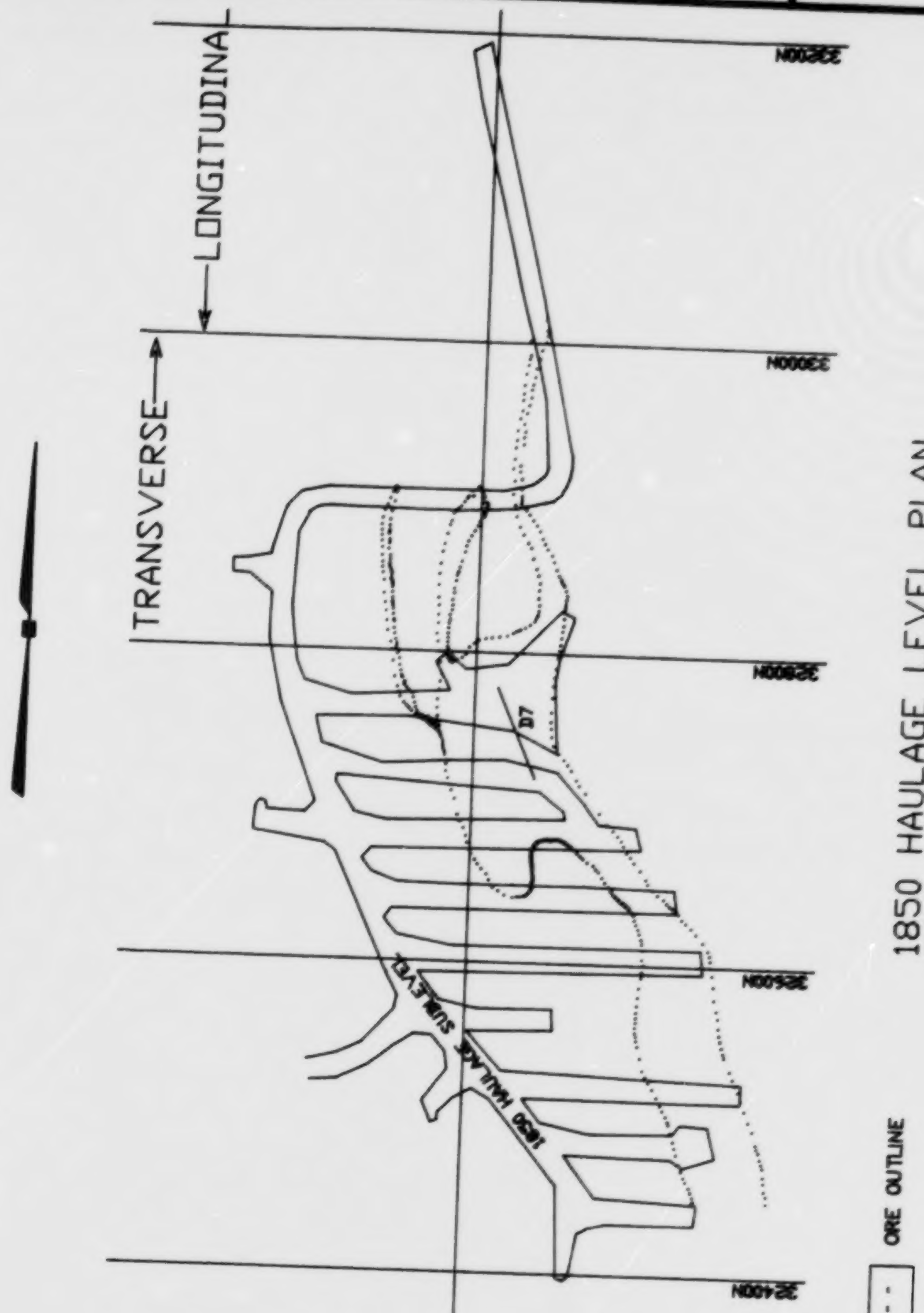
X - SECTION TRANSVERSE BLOCK


TOP SILL
CEMENTED FILL

LONG SECTION TRANSVERSE BLOCK

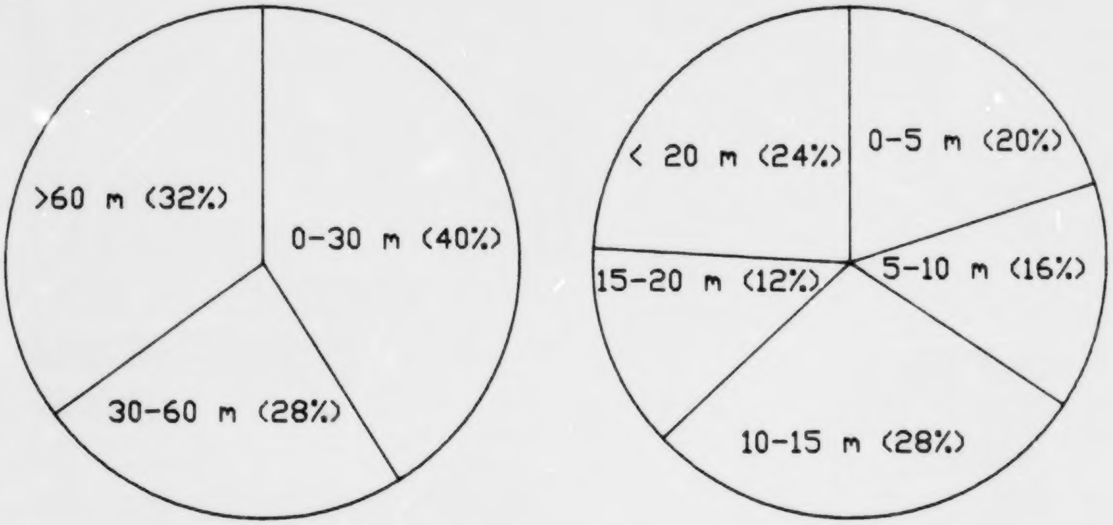
PLAN OF TRANSVERSE AND LONGITUDINAL MINING

FIG. 3.4

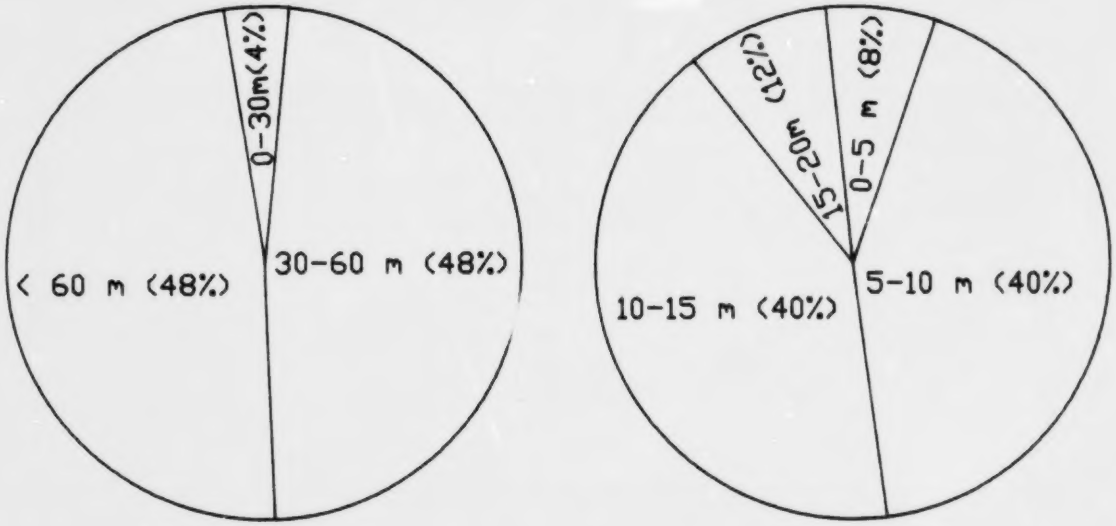

LONGITUDINA
TRANSVERSE
33200N
33000N
32800N
32600N
32400N
10400E
D7
1850 HAULAGE SUBLEVEL
1850 HAULAGE LEVEL PLAN
ORE OUTLINE

SIZE OF OPEN STOPES IN CANADA (PAKALNIS)

FIG. 3.5


OPEN STOPE DIMENSIONS-SPANS
DATA BASE-25 STOPES (15 MINES)

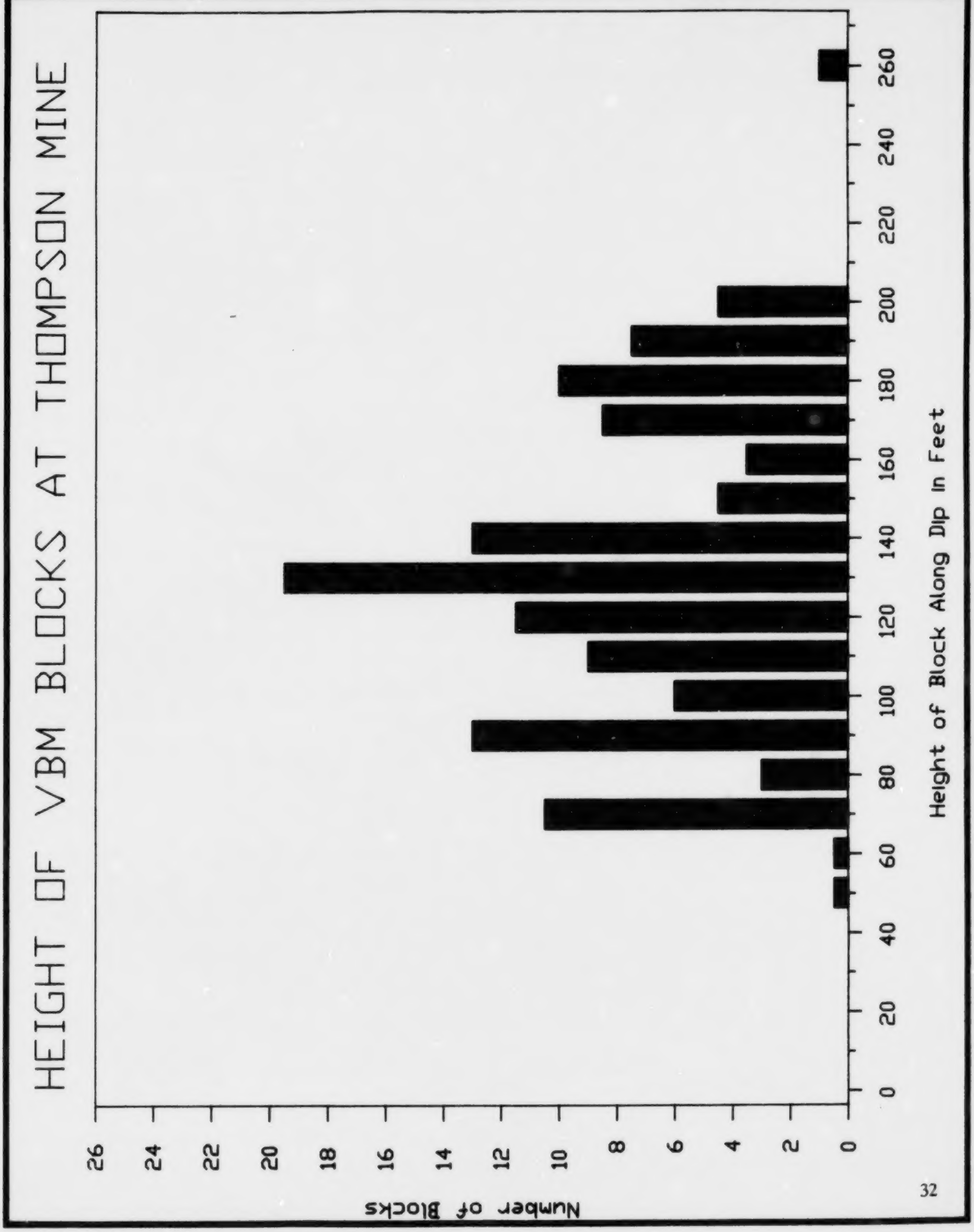
OPEN STOPE DIMENSIONS-WIDTHS
DATA BASE-25 STOPES (15 MINES)


OPEN STOPE DIMENSIONS-HEIGHTS
DATA BASE-25 STOPES (15 MINES)

HYDR. RADIUS-OPEN STOPES
DATA BASE-25 STOPES (15 MINES)

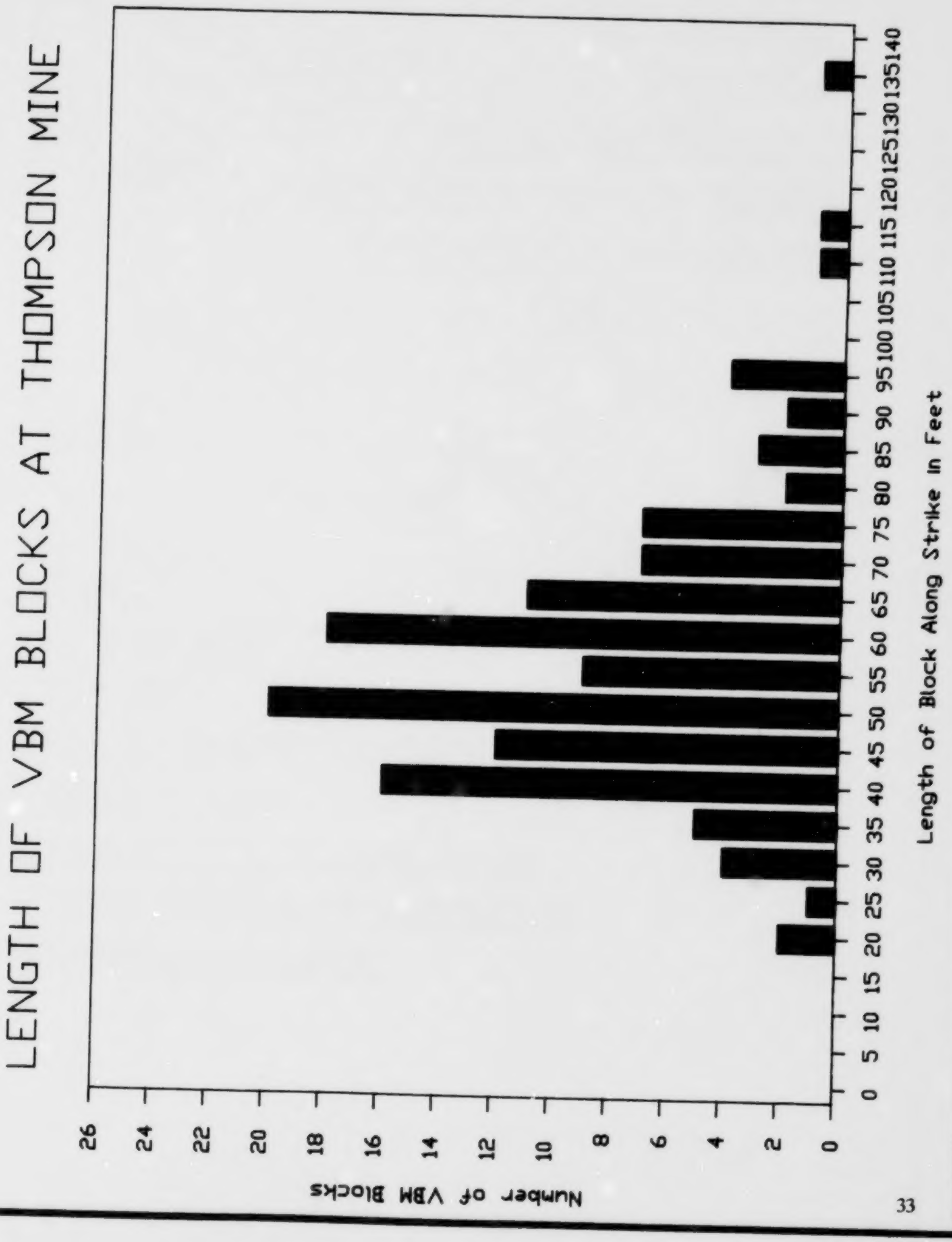
HEIGHT OF VBM BLOCKS
AT THOMPSON MINE

FIG. 3.6



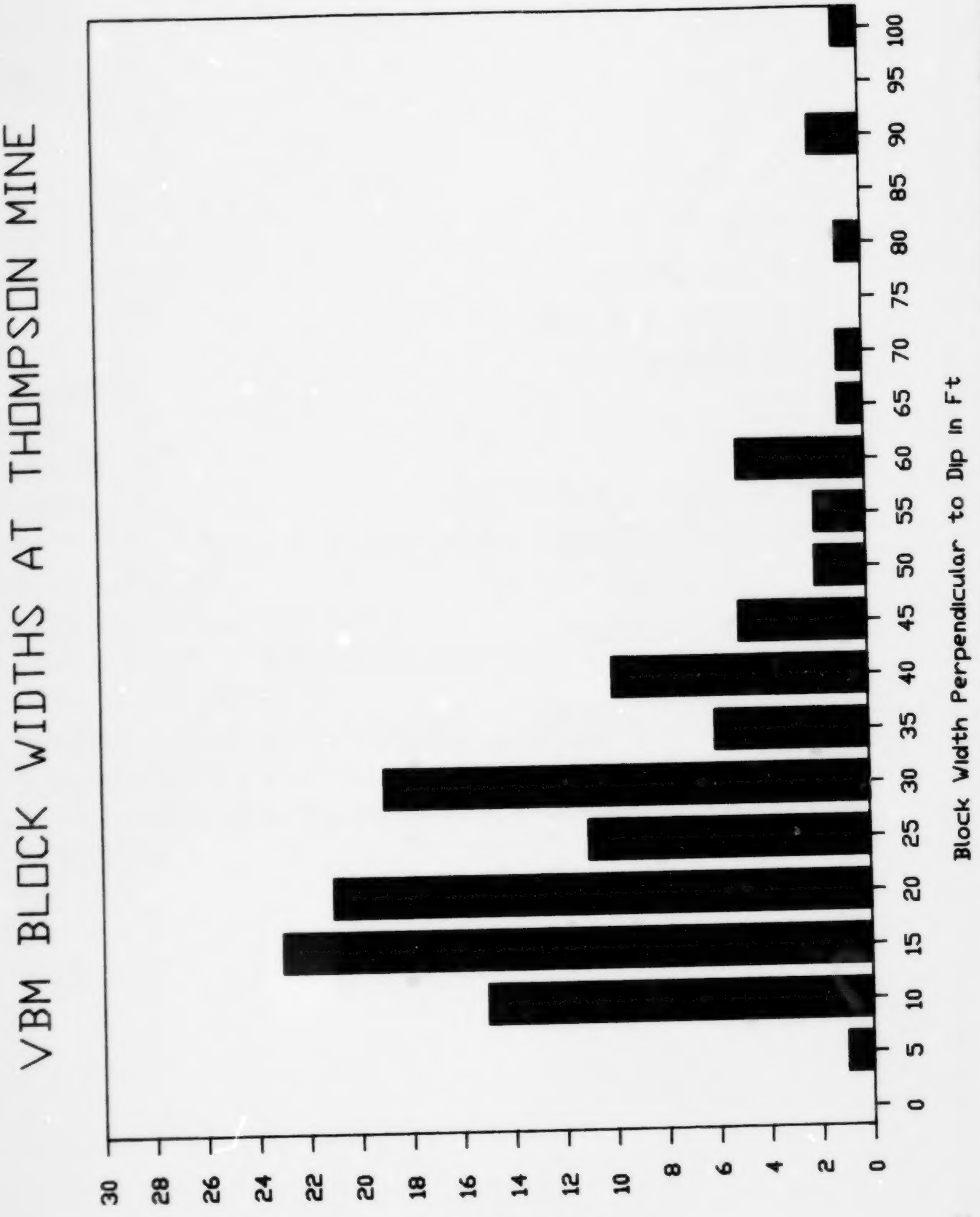
LENGTH OF VBM BLOCKS AT THOMPSON MINE

FIG. 3.7



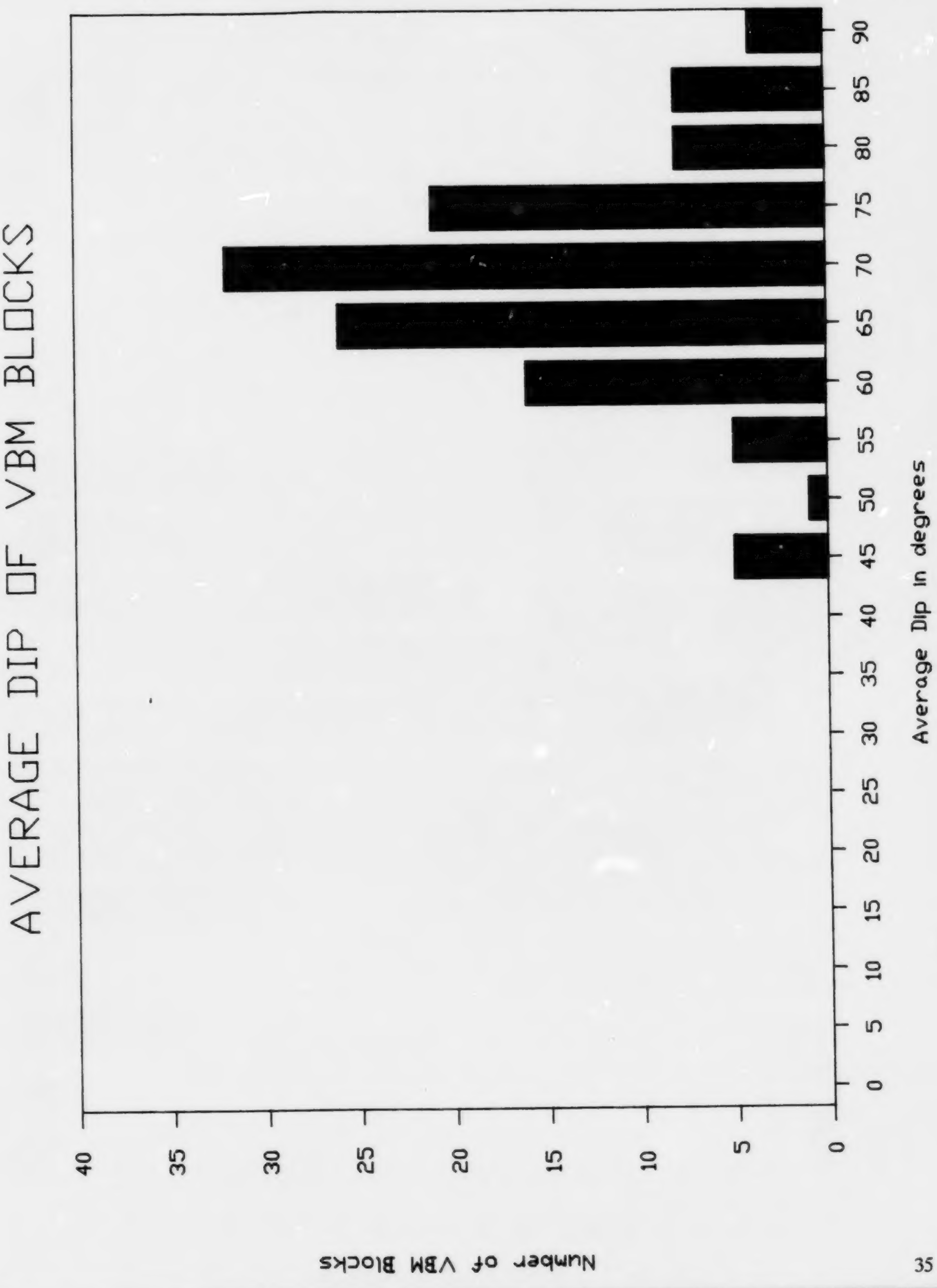
VBM BLOCK WIDTHS AT THOMPSON MINE

FIG. 3.8



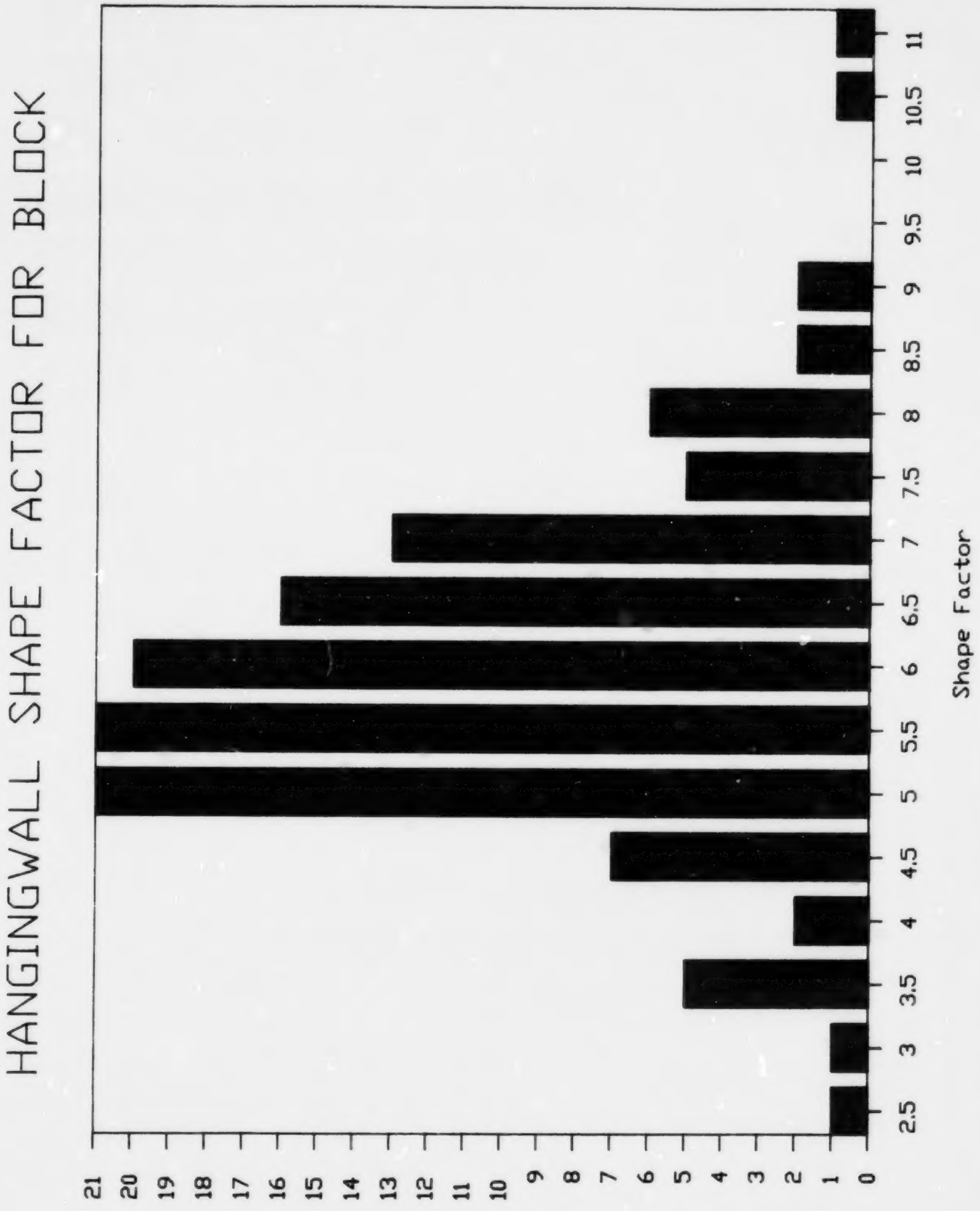
AVERAGE DIP OF VBM BLOCK

FIG. 3.9



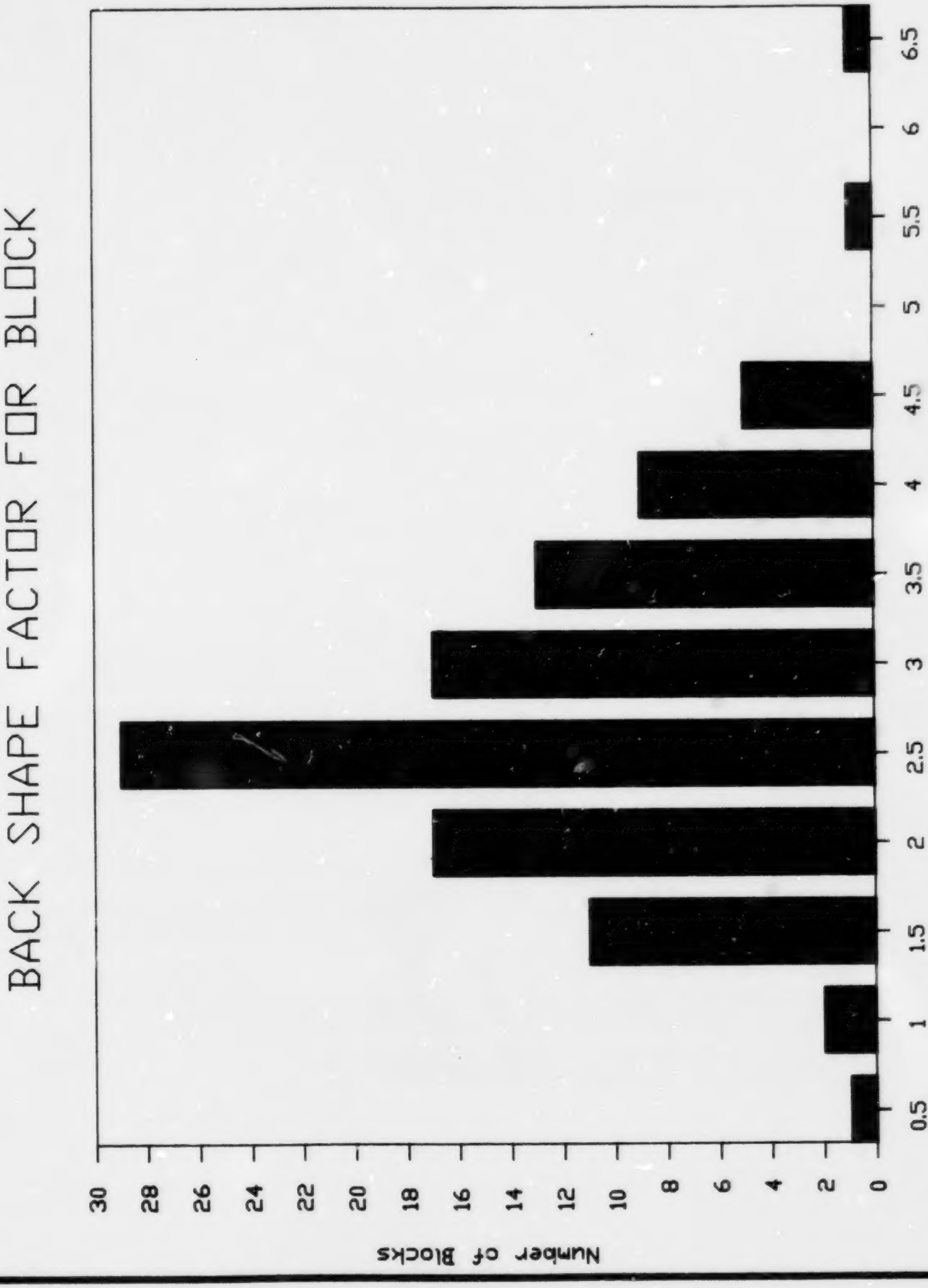
HANGINGWALL SHAPE FACTOR FOR BLOCK

FIG. 3.10



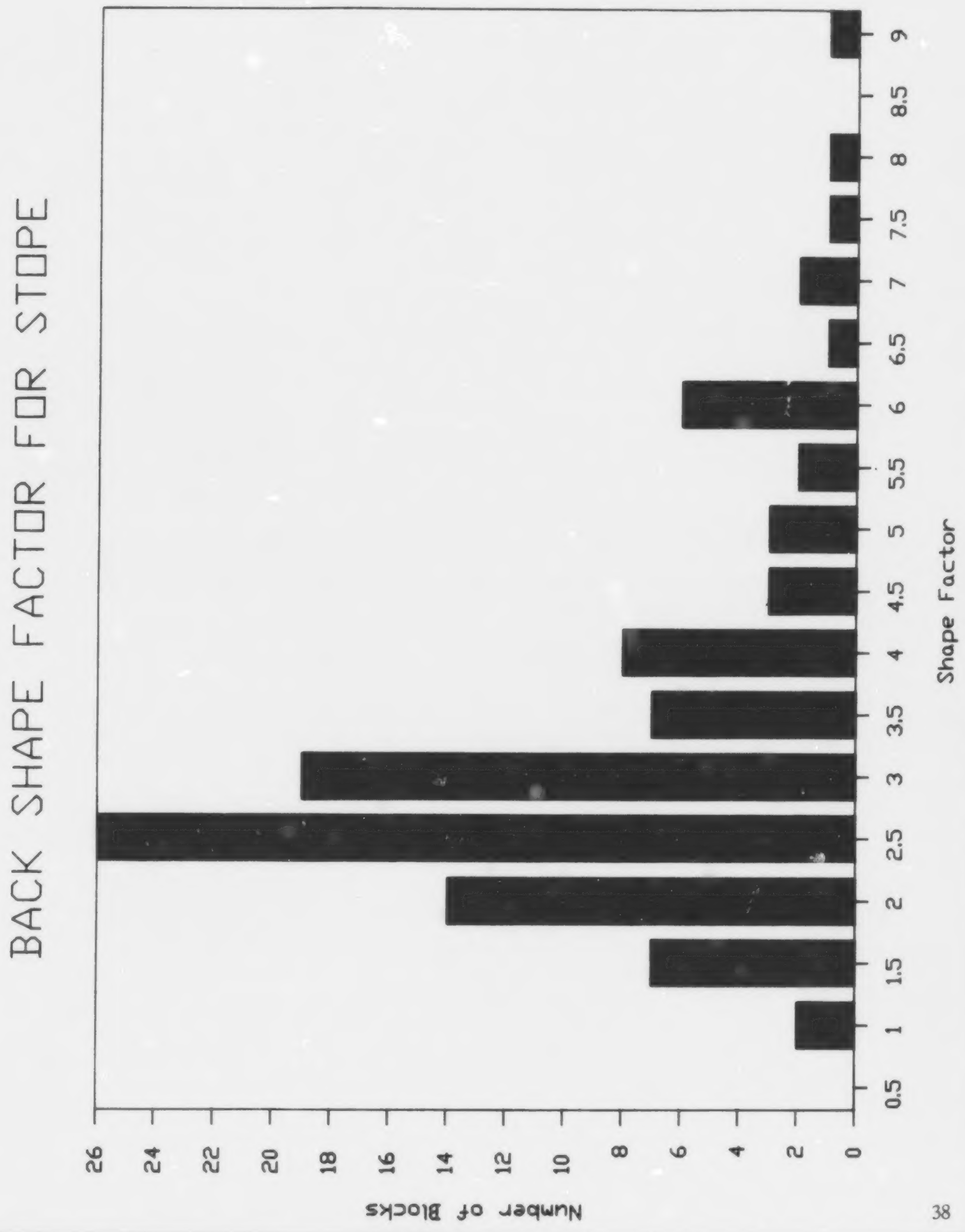
BACK SHAPE FACTOR FOR BLOCK

FIG. 3.11



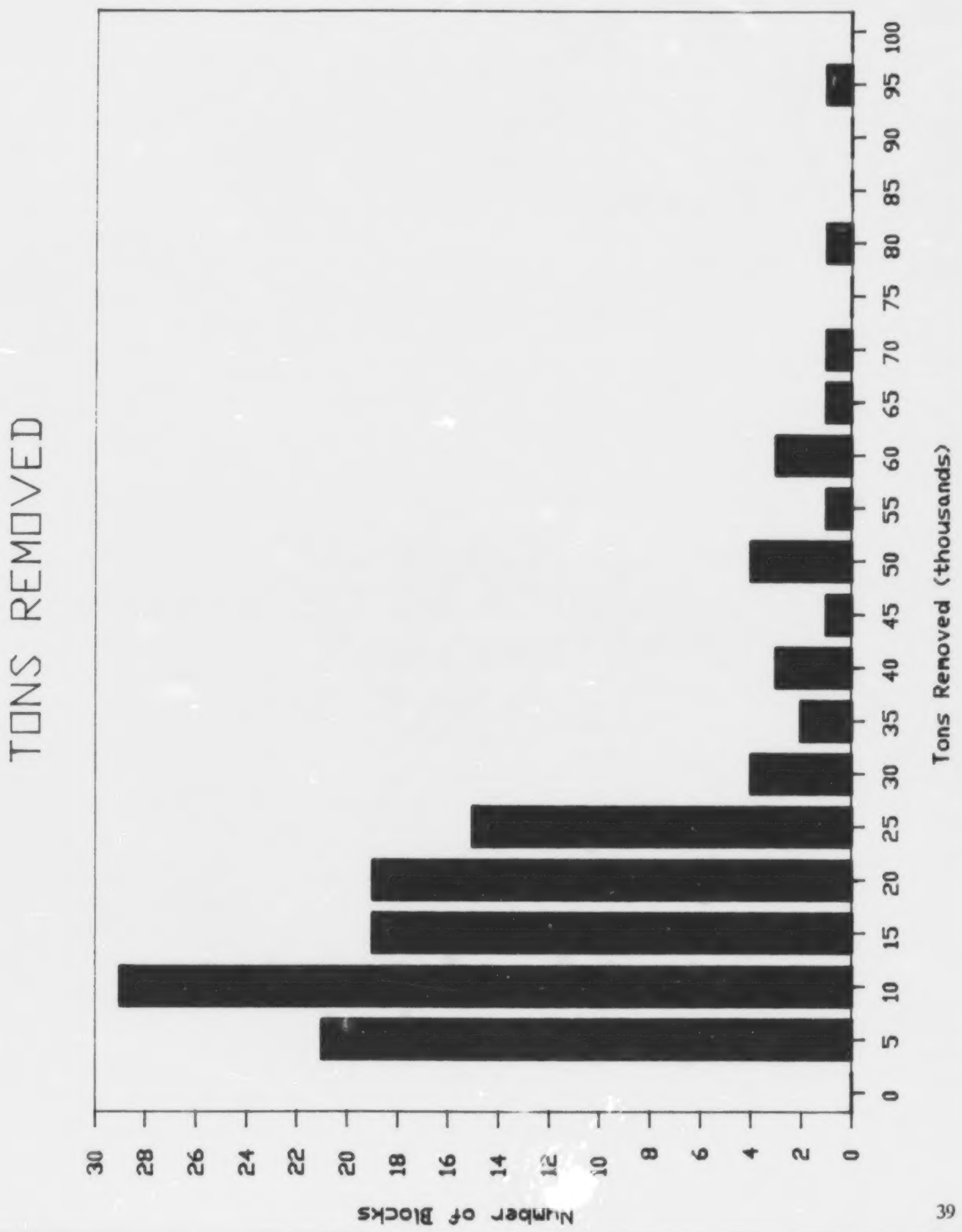
BACK SHAPE FACTOR FOR STOPE

FIG. 3.12



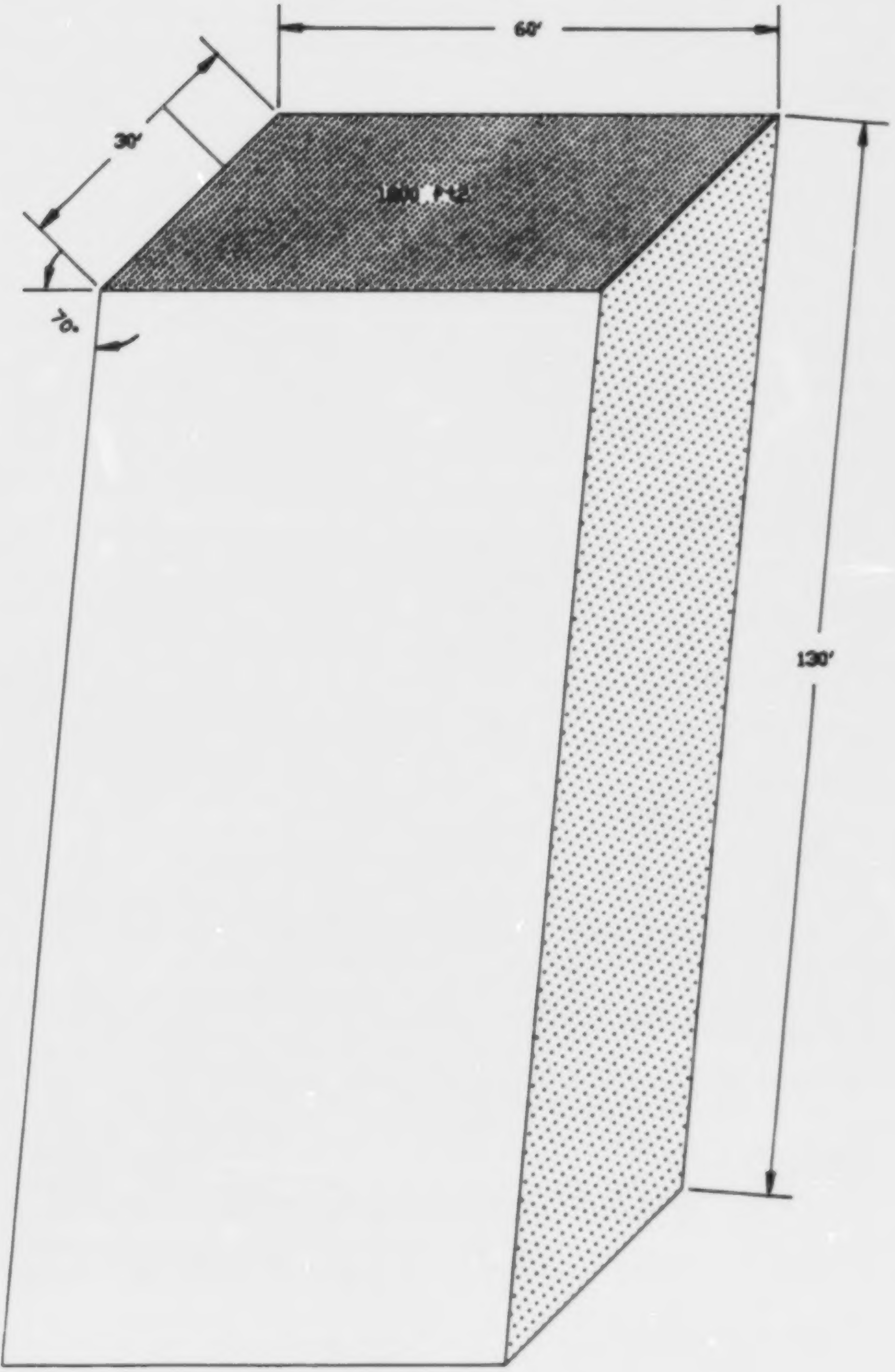
TONS REMOVED

FIG. 3.13



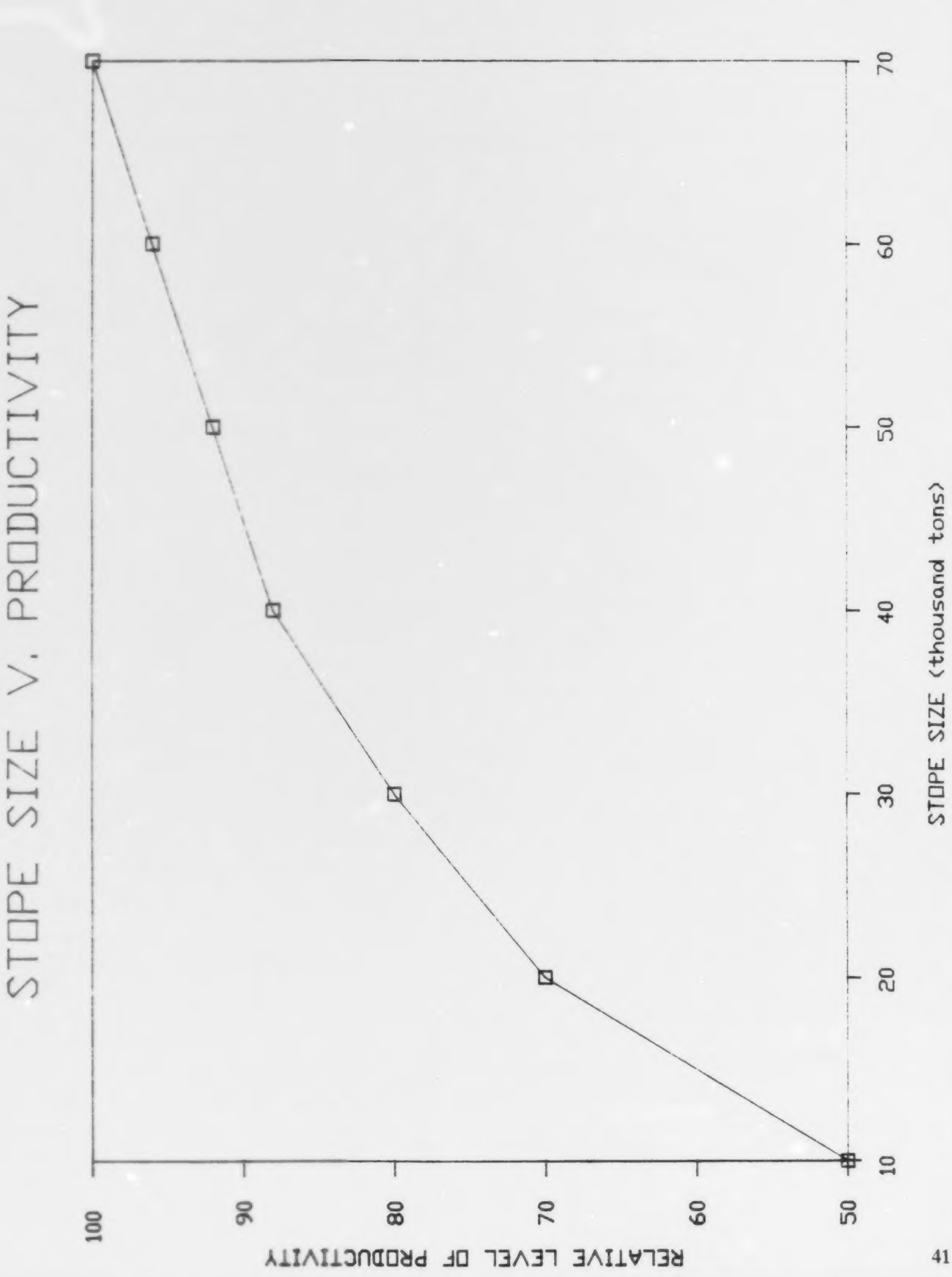
TYPICAL OPEN STOPE DIMENSIONS AT THOMPSON MINE

FIG. 3.14


60'
30'
1800 FT2
70°
130'

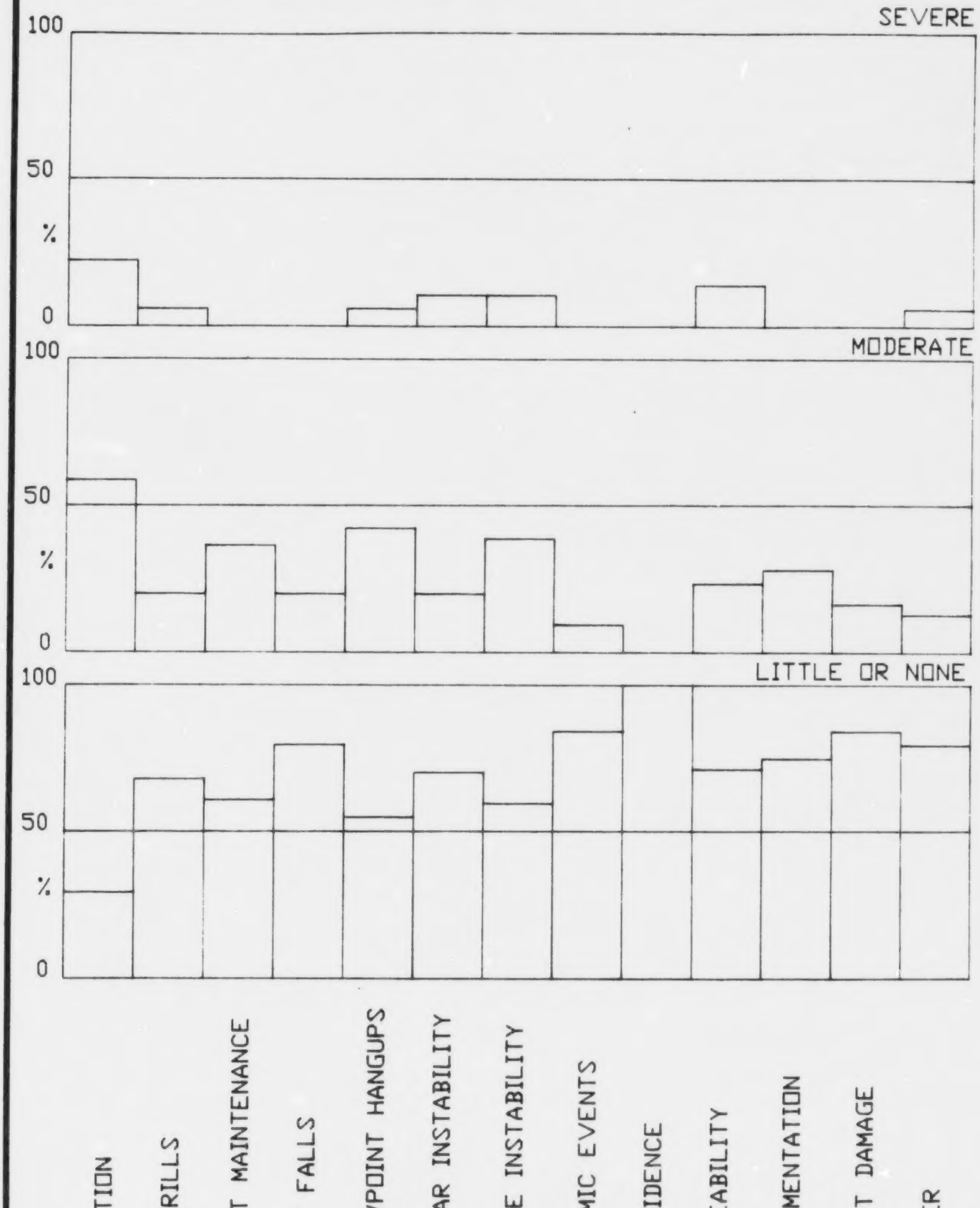
STOPE SIZE V. PRODUCTIVITY

FIG. 3.15



GROUND CONTROL PROBLEMS IN CANADIAN MINES

FIG. 3.16



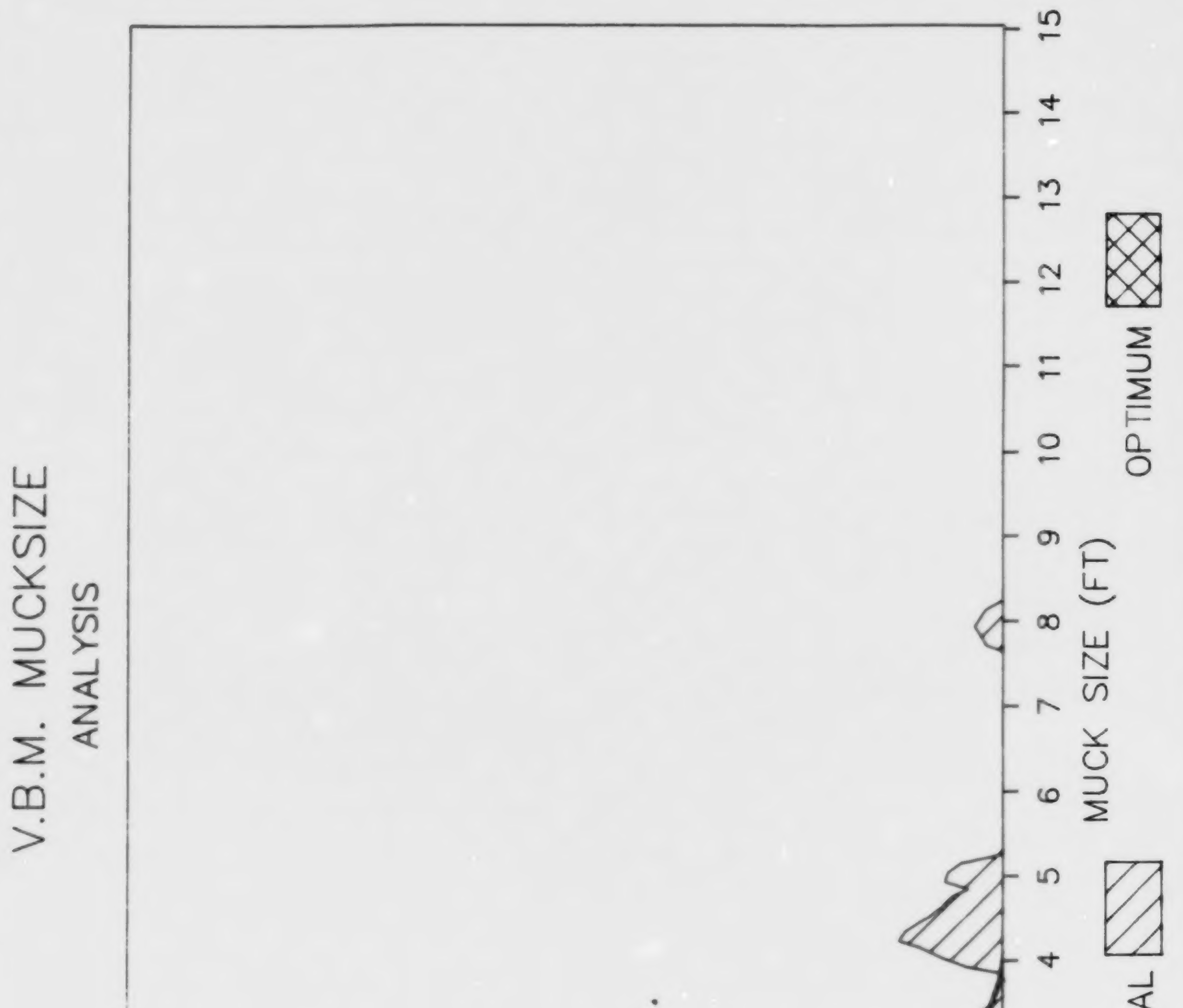
GROUND CONTROL PROBLEMS IN OPEN STOPING OPERATIONS AT THOMPSON MINE

FIG. 3.17

	LITTLE OR NONE	MODERATE	SEVERE
DILUTION			✓
RE-DRILLS		✓	
DRIFT MAINTENANCE		✓	
ROOF FALLS		✓	
DRAWPOINT HANGUPS		✓	
PILLAR INSTABILITY	✓		
STOPE INSTABILITY		✓	
SEISMIC EVENTS	✓		
SUBSIDENCE	✓		
CAVEABILITY		✓	
FRAGMENTATION		✓	
BLAST DAMAGE			✓
WATER		✓	

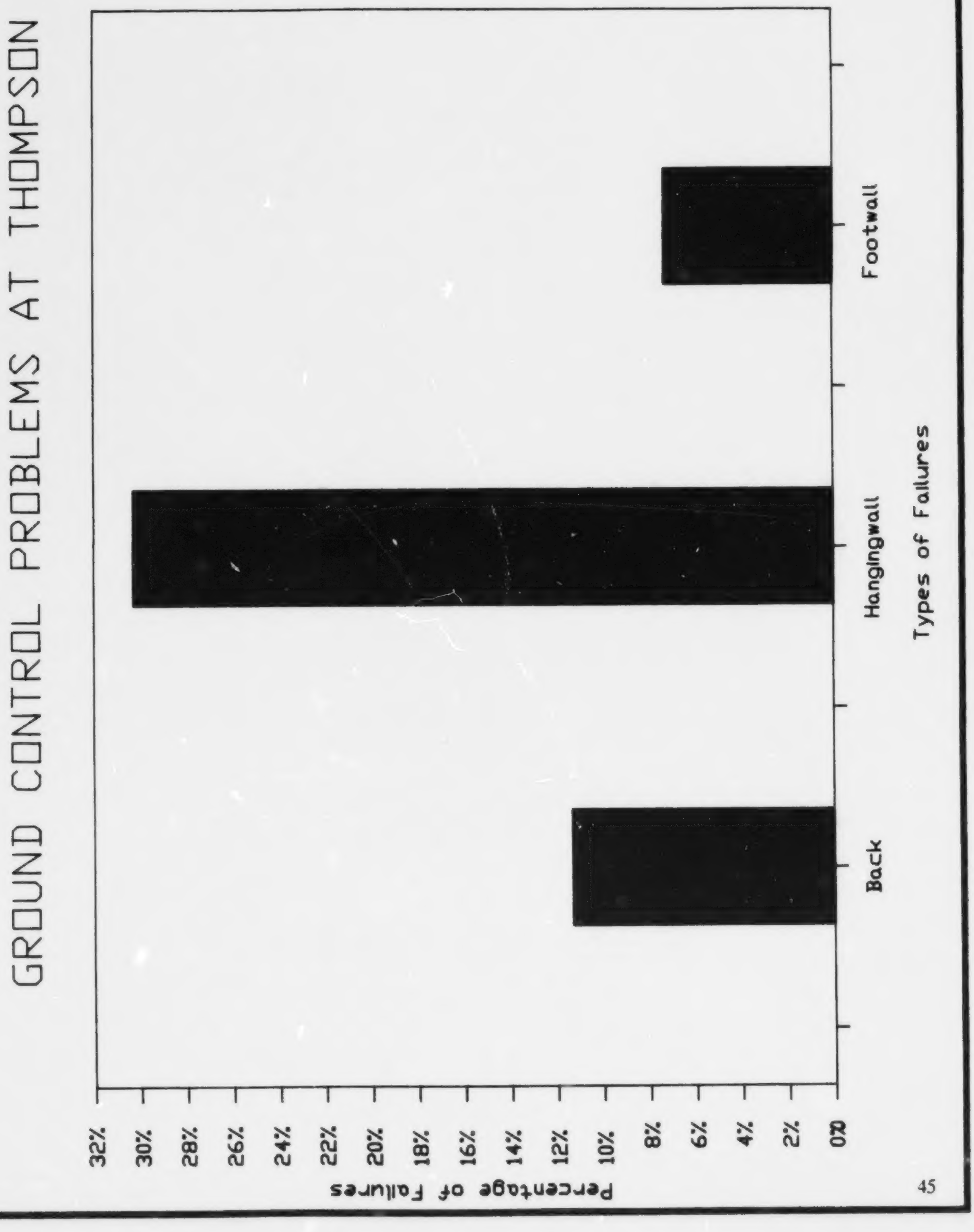
V.B.M. MUCKSIZE ANALYSIS AT THOMPSON MINE

FIG. 3.18



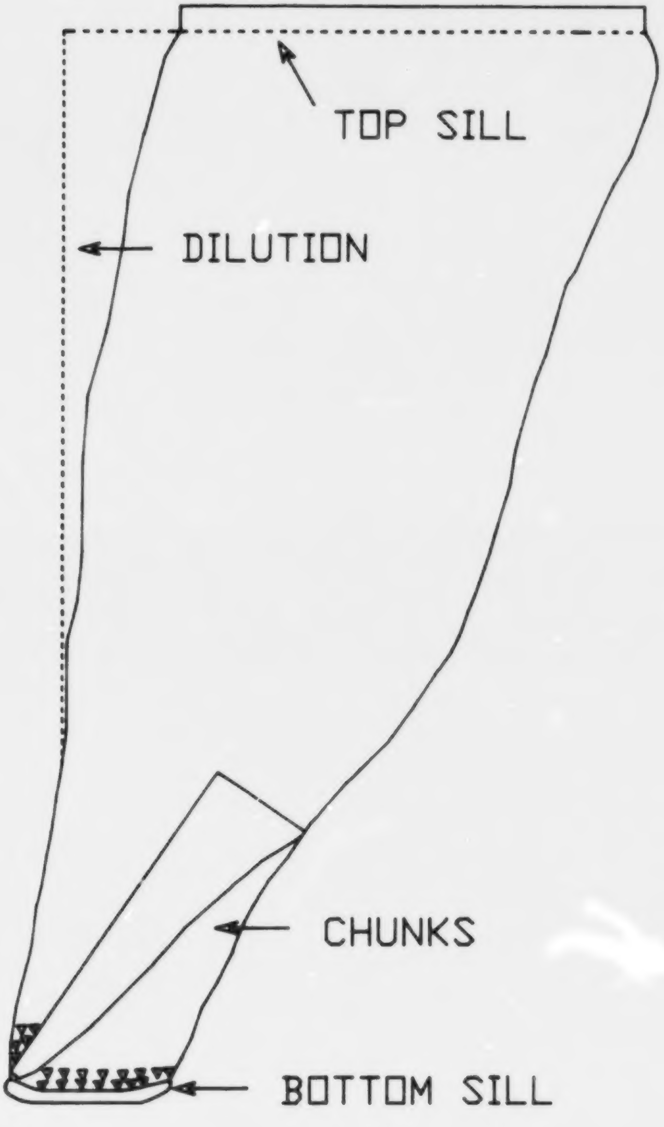
GROUND CONTROL PROBLEMS AT THOMPSON MINE

FIG. 3.19



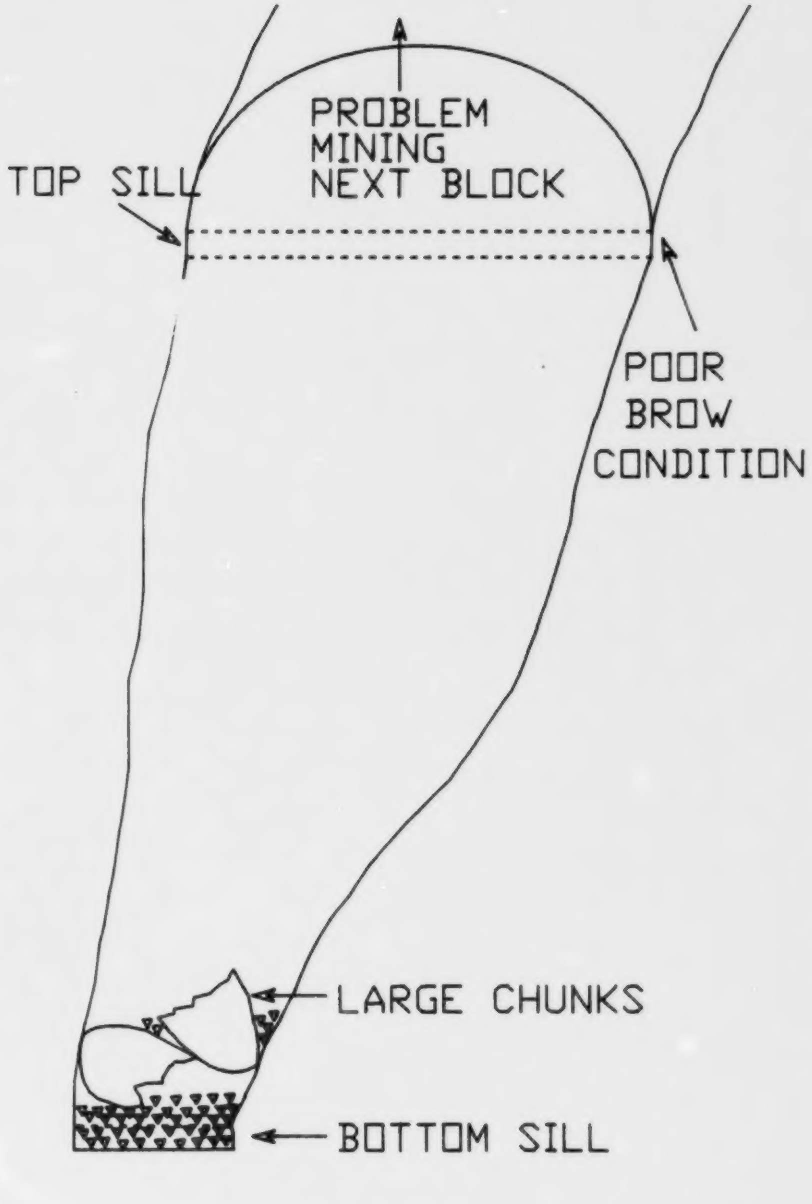
IMPACT OF HANGING WALL FAILURE

FIG. 3.20


TOP SILL
DILUTION
CHUNKS
BOTTOM SILL

IMPACT OF BACK FAILURE

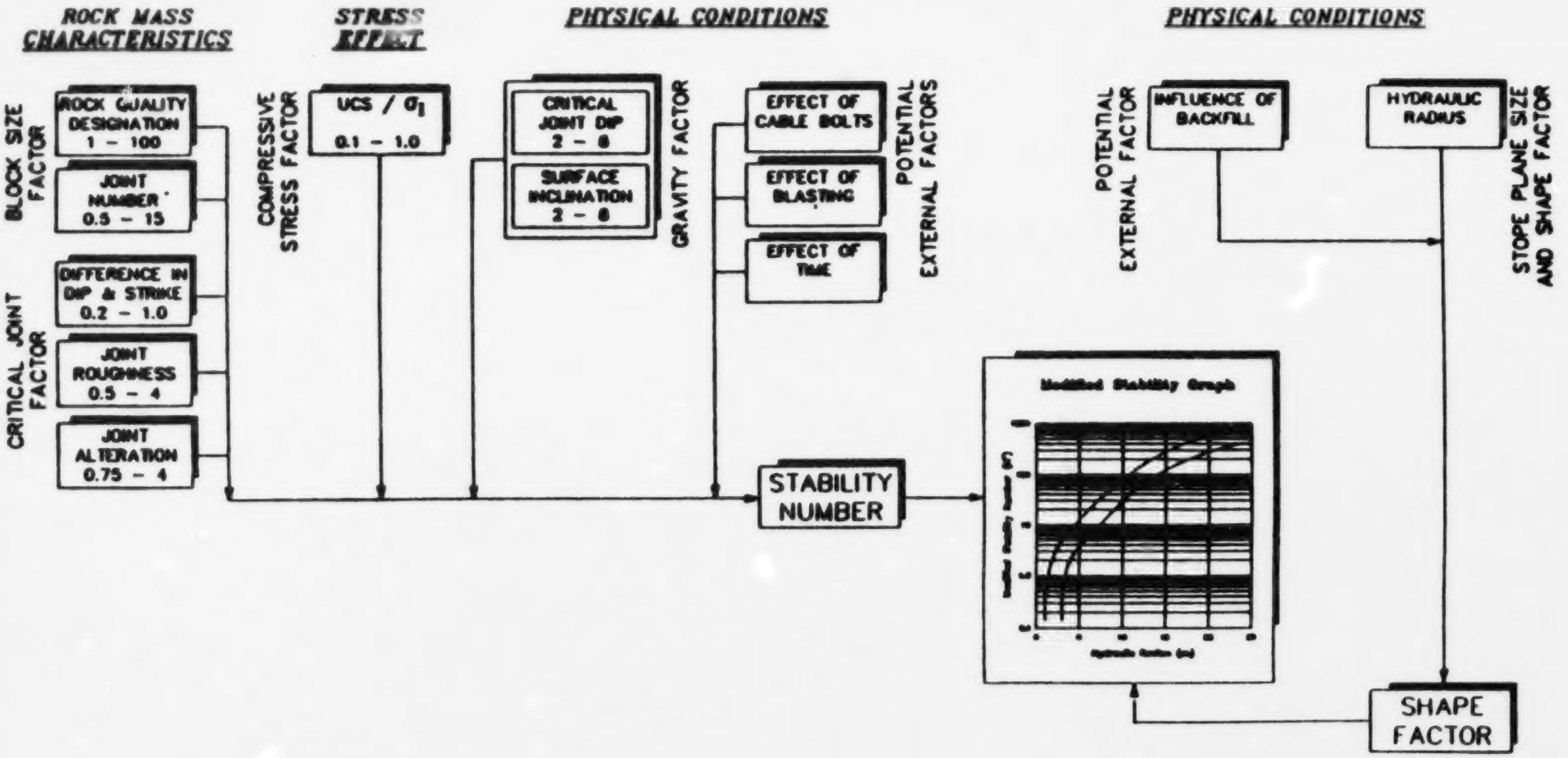
FIG. 3.21


PROBLEM
MINING
NEXT BLOCK
TOP SILL
POOR
BROW
CONDITION
LARGE CHUNKS
BOTTOM SILL

PREDICTION MODEL

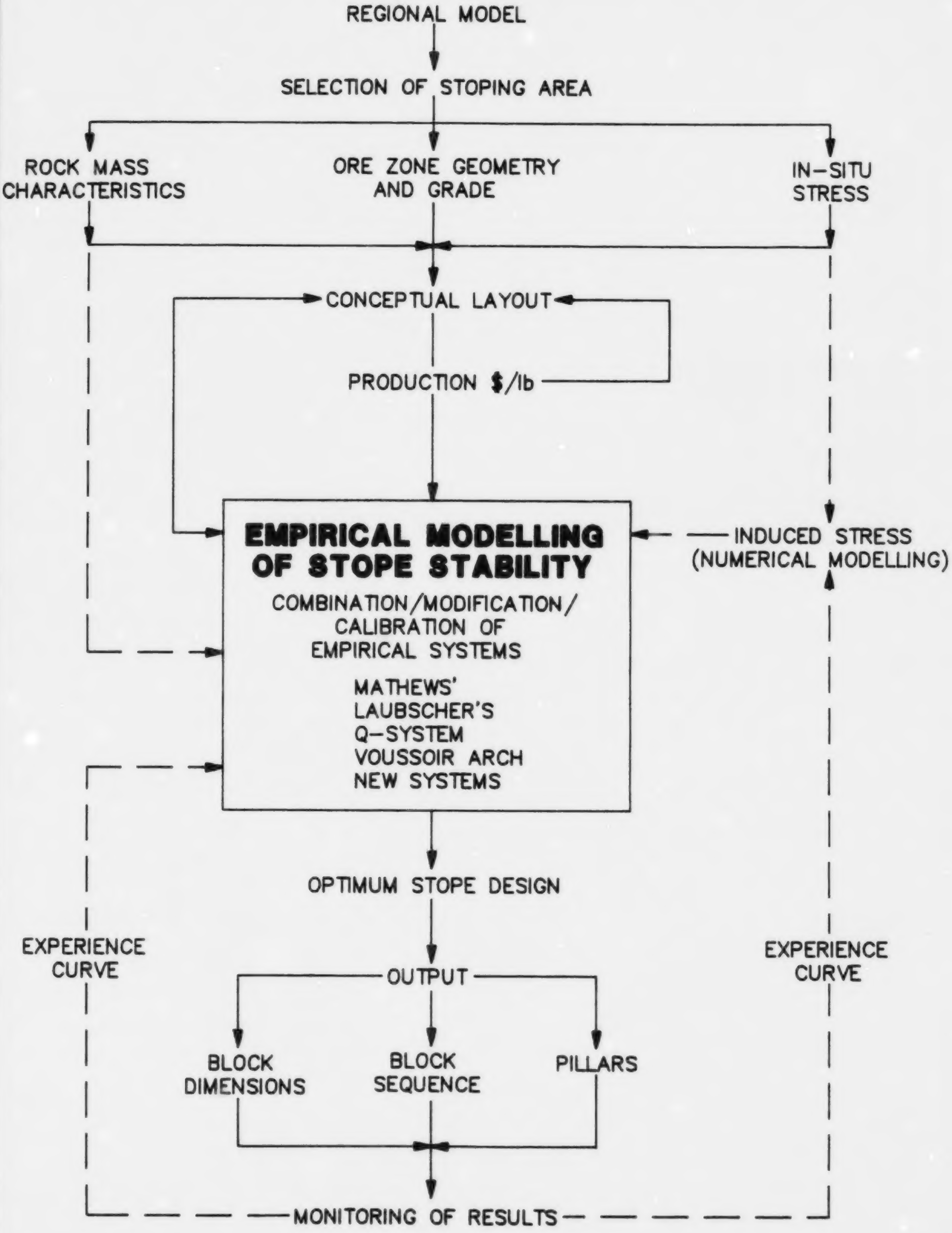
FIG. 3.22

STABILITY GRAPH METHOD FOR OPEN STOPE DESIGN


ROCK MASS CHARACTERISTICS
STRESS EFFECT
PHYSICAL CONDITIONS
PHYSICAL CONDITIONS
BLOCK SIZE FACTOR
ROCK QUALITY DESIGNATION 1 – 100
JOINT NUMBER 0.5 – 15
CRITICAL JOINT FACTOR
DIFFERENCE IN DIP & STRIKE 0.2 – 1.0
JOINT ROUGHNESS 0.5 – 4
JOINT ALTERATION 0.75 – 4
COMPRESSIVE STRESS FACTOR
UCS / σ_1 0.1 – 1.0
CRITICAL JOINT DIP 2 – 8
SURFACE INCLINATION 2 – 8
GRAVITY FACTOR
EFFECT OF CABLE BOLTS
EFFECT OF BLASTING
EFFECT OF TIME
POTENTIAL EXTERNAL FACTORS
STABILITY NUMBER
Modified Stability Graph
POTENTIAL EXTERNAL FACTOR
INFLUENCE OF BACKFILL
HYDRAULIC RADIUS
STOPE PLANE SIZE AND SHAPE FACTOR
SHAPE FACTOR

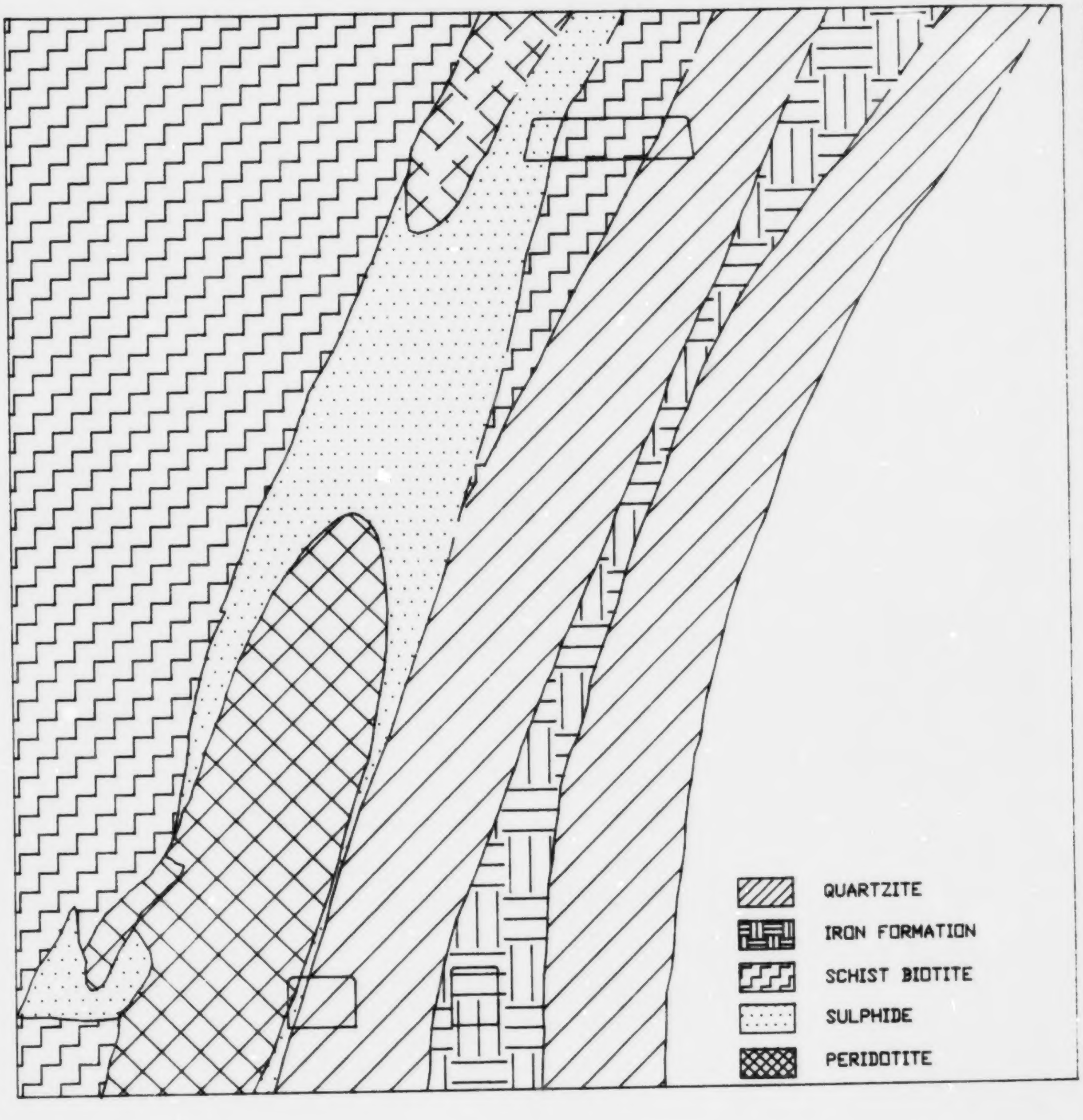
STOPE DESIGN FLOWSHEET

FIG. 3.23


REGIONAL MODEL
SELECTION OF STOPING AREA
ROCK MASS CHARACTERISTICS
ORE ZONE GEOMETRY AND GRADE
IN-SITU STRESS
CONCEPTUAL LAYOUT
PRODUCTION $/lb
EMPIRICAL MODELLING OF STOPE STABILITY
COMBINATION/MODIFICATION/ CALIBRATION OF EMPIRICAL SYSTEMS
MATHEWS'
LAUBSCHER'S
Q-SYSTEM
VOUSSOIR ARCH
NEW SYSTEMS
INDUCED STRESS (NUMERICAL MODELLING)
OPTIMUM STOPE DESIGN
EXPERIENCE CURVE
EXPERIENCE CURVE
OUTPUT
BLOCK DIMENSIONS
BLOCK SEQUENCE
PILLARS
MONITORING OF RESULTS

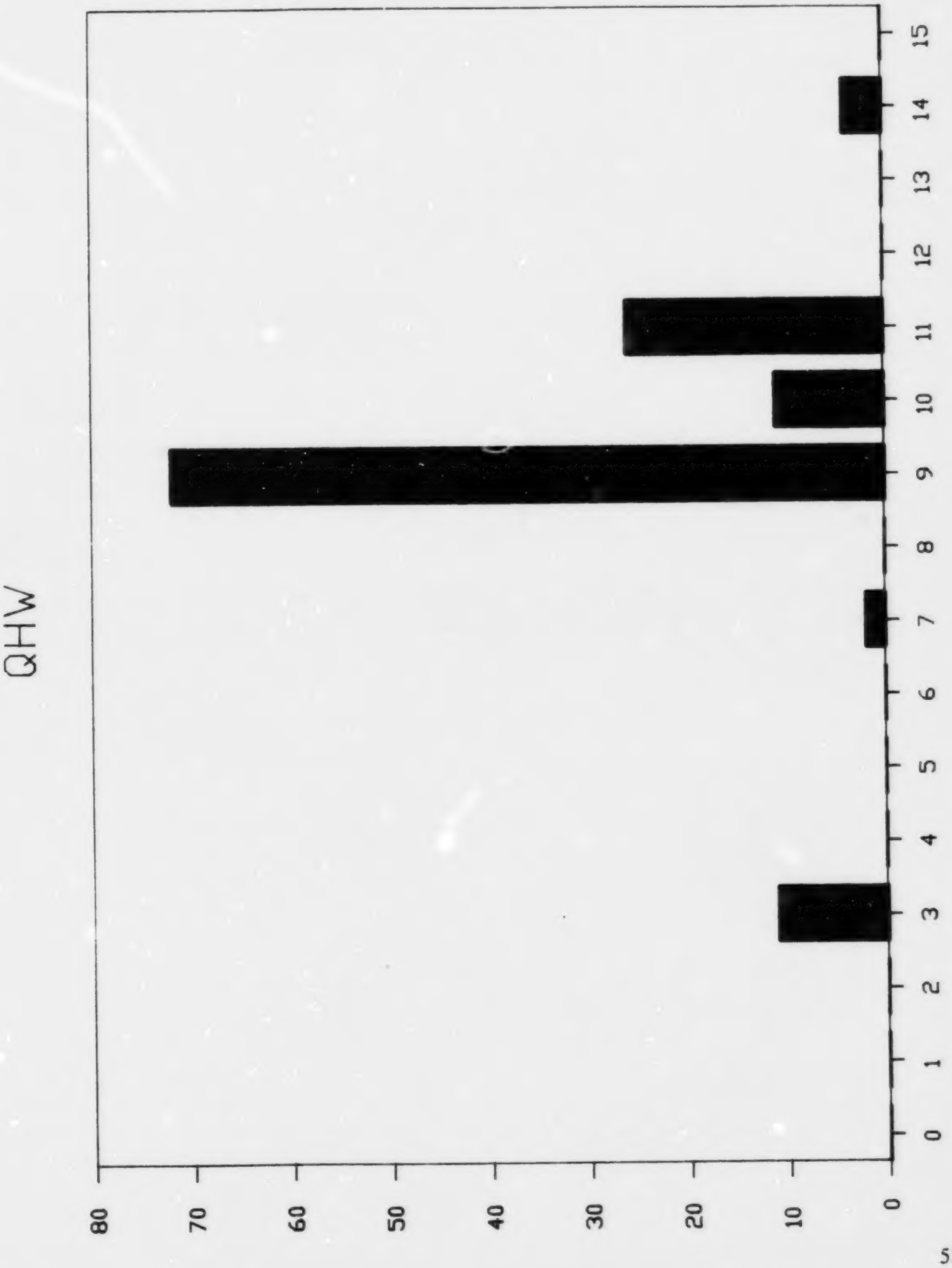
TYPICAL LITHOLOGY AT THOMPSON MINE

FIG. 3.24


▨ QUARTZITE
IRON FORMATION
SCHIST BIOTITE
SULPHIDE
PERIDOTITE

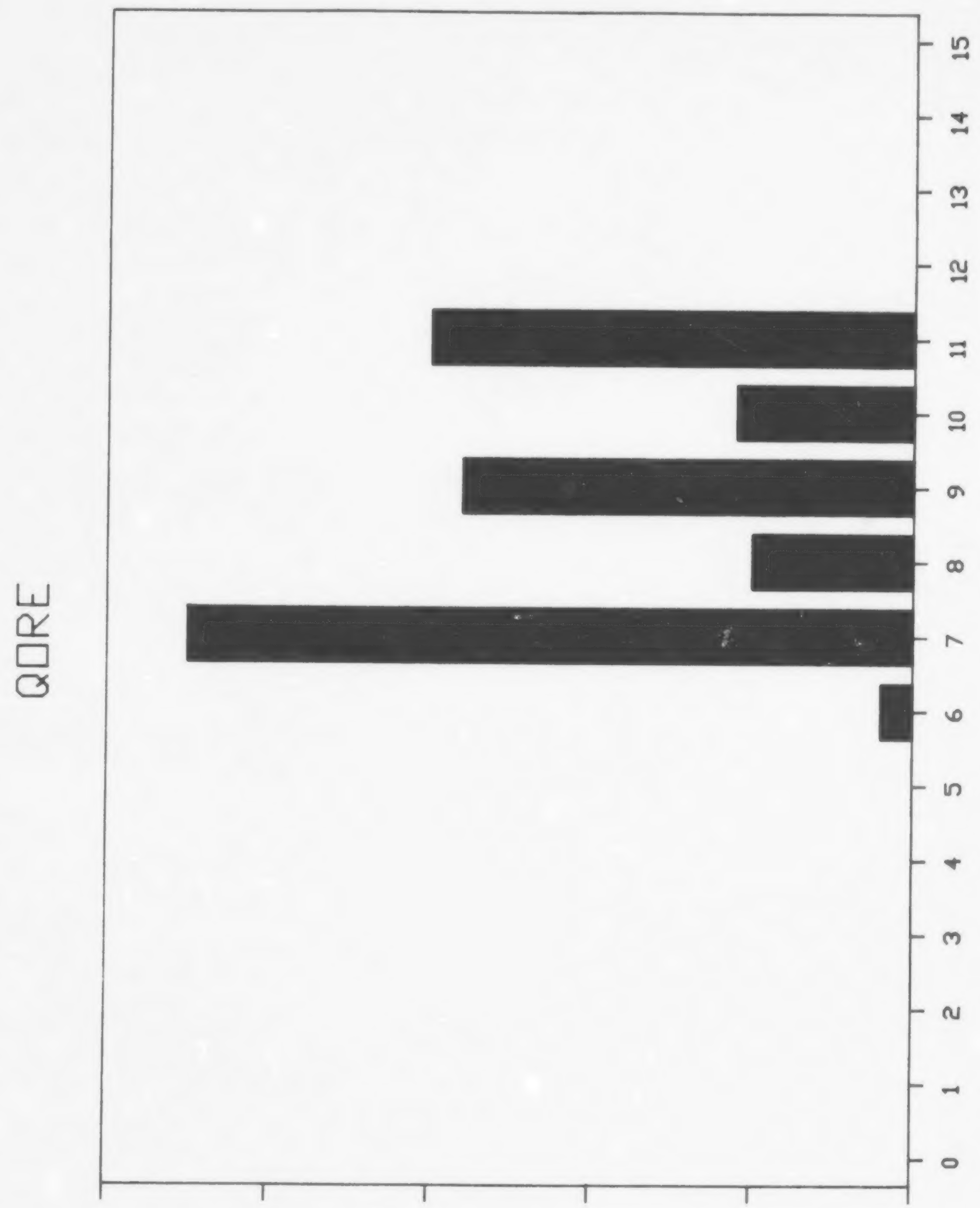
HANGINGWALL QUALITY

FIG. 3.25



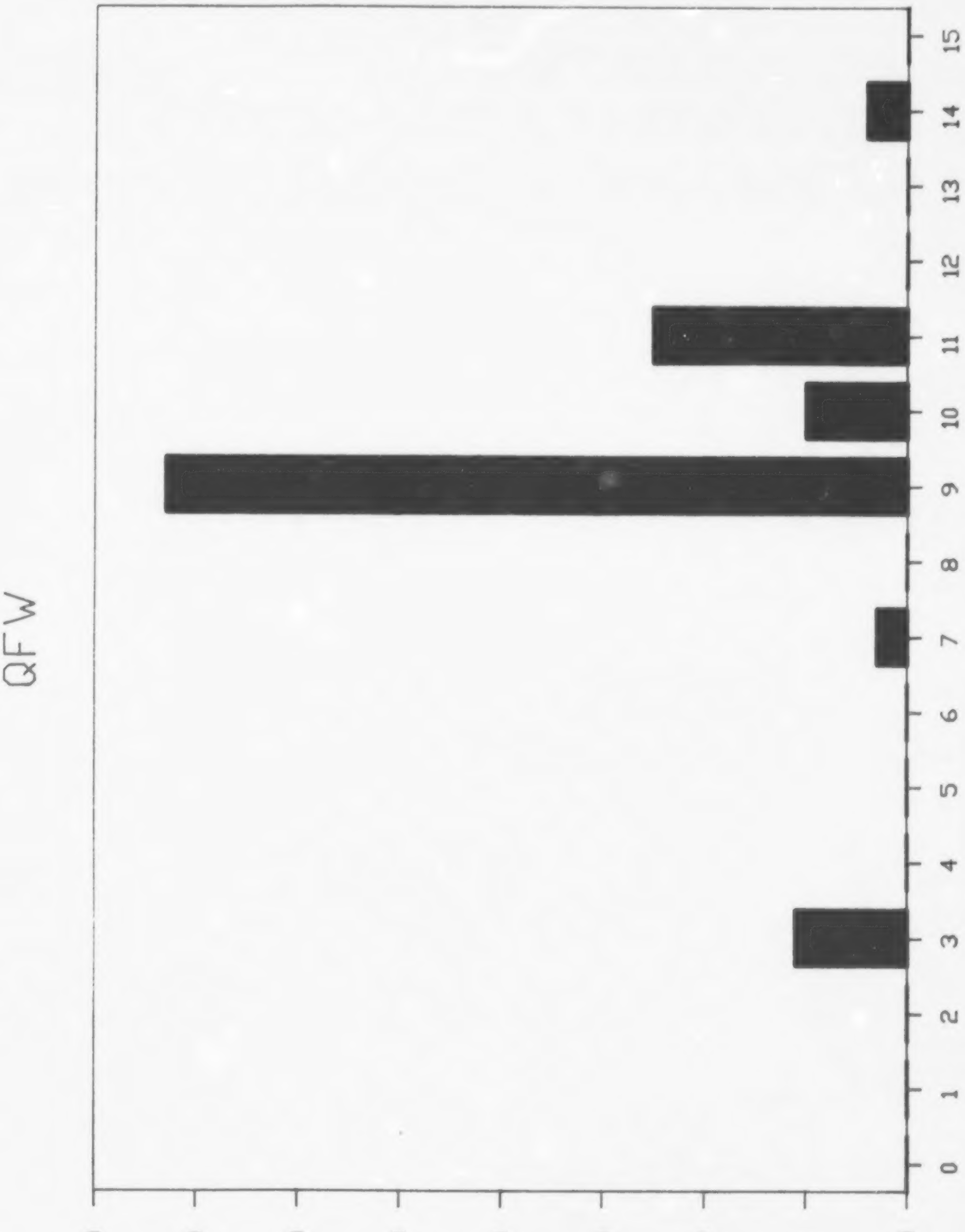
ORE QUALITY

FIG. 3.26



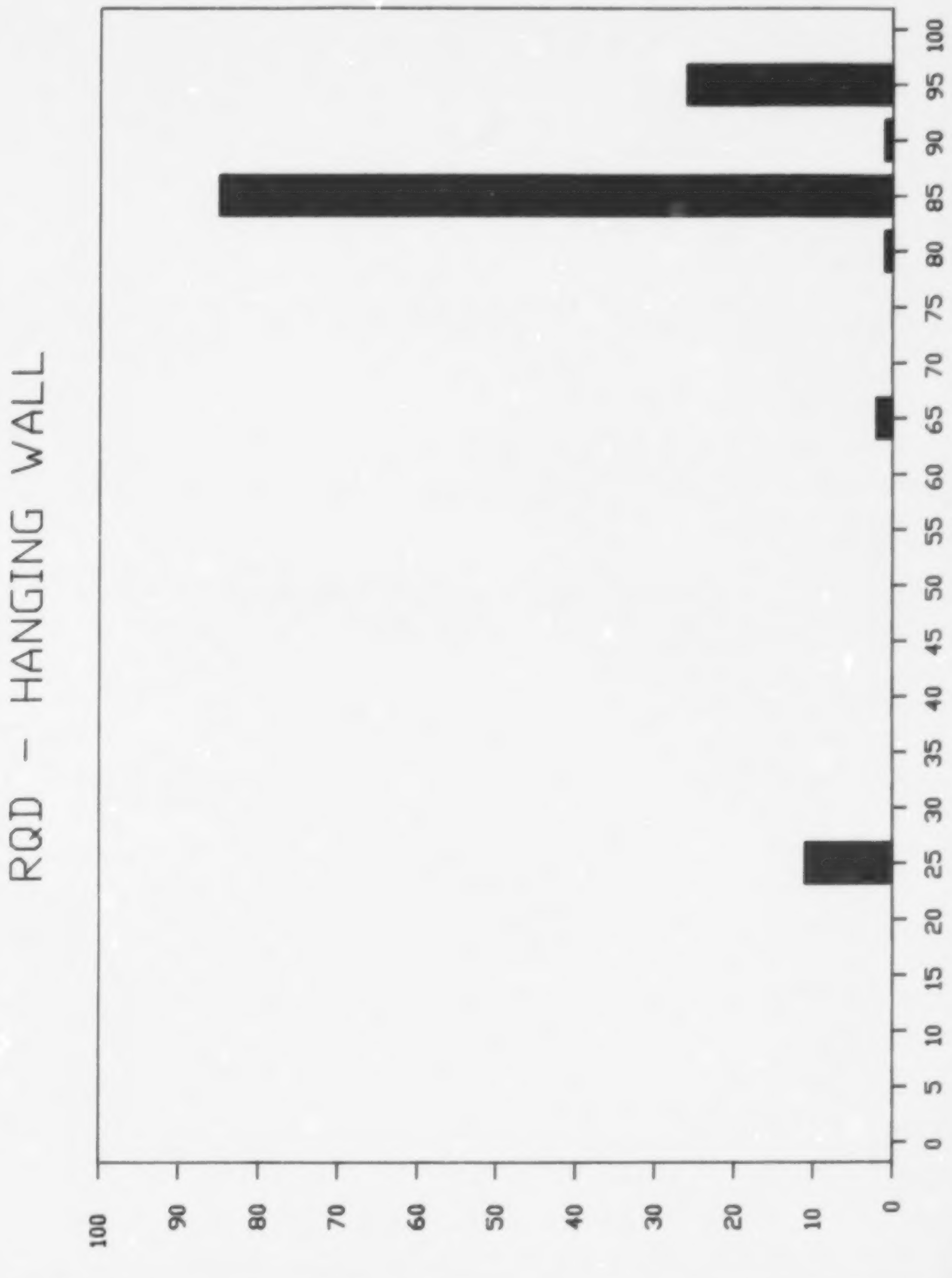
FOOTWALL QUALITY

FIG. 3.27



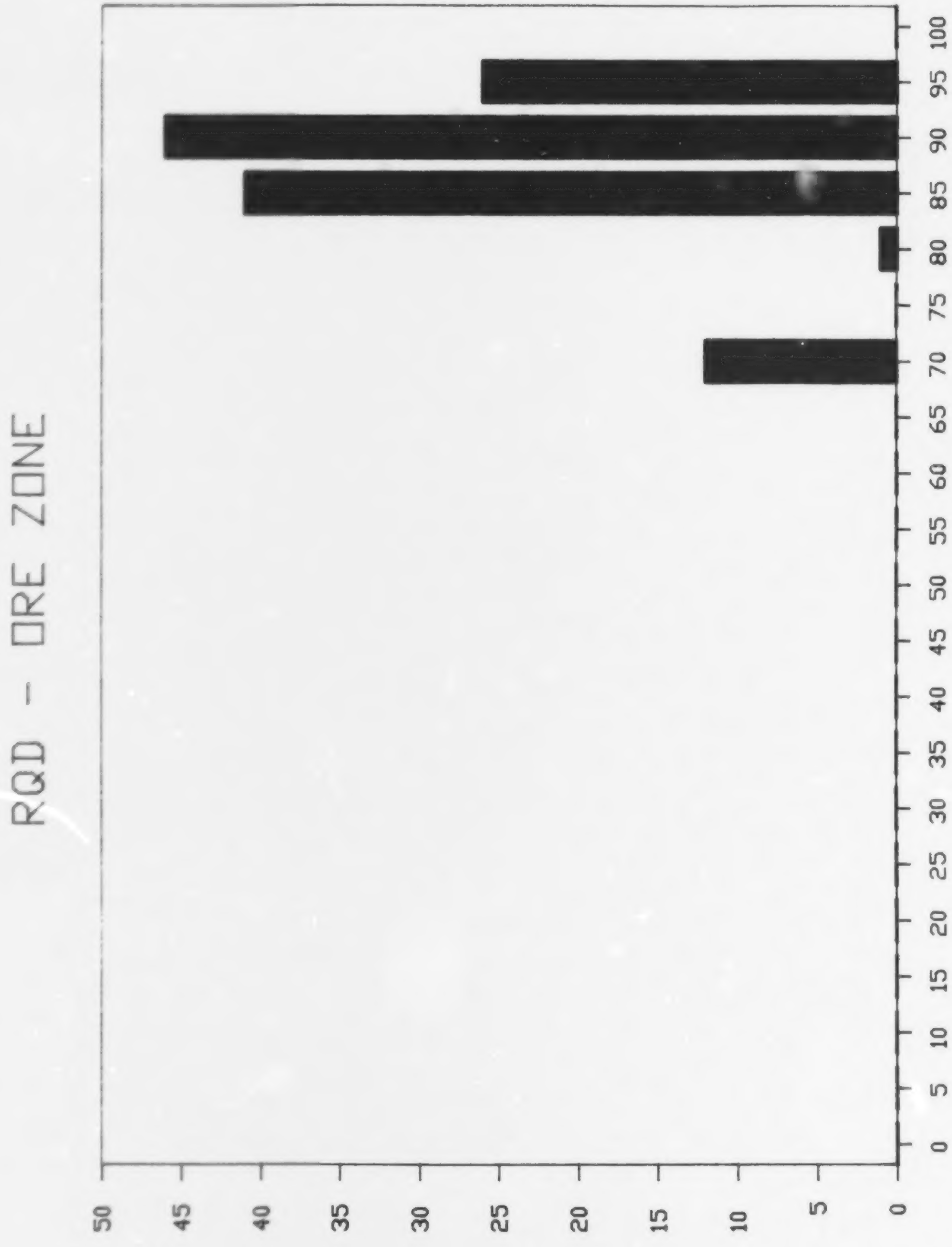
RQD — HANGINGWALL

FIG. 3.28



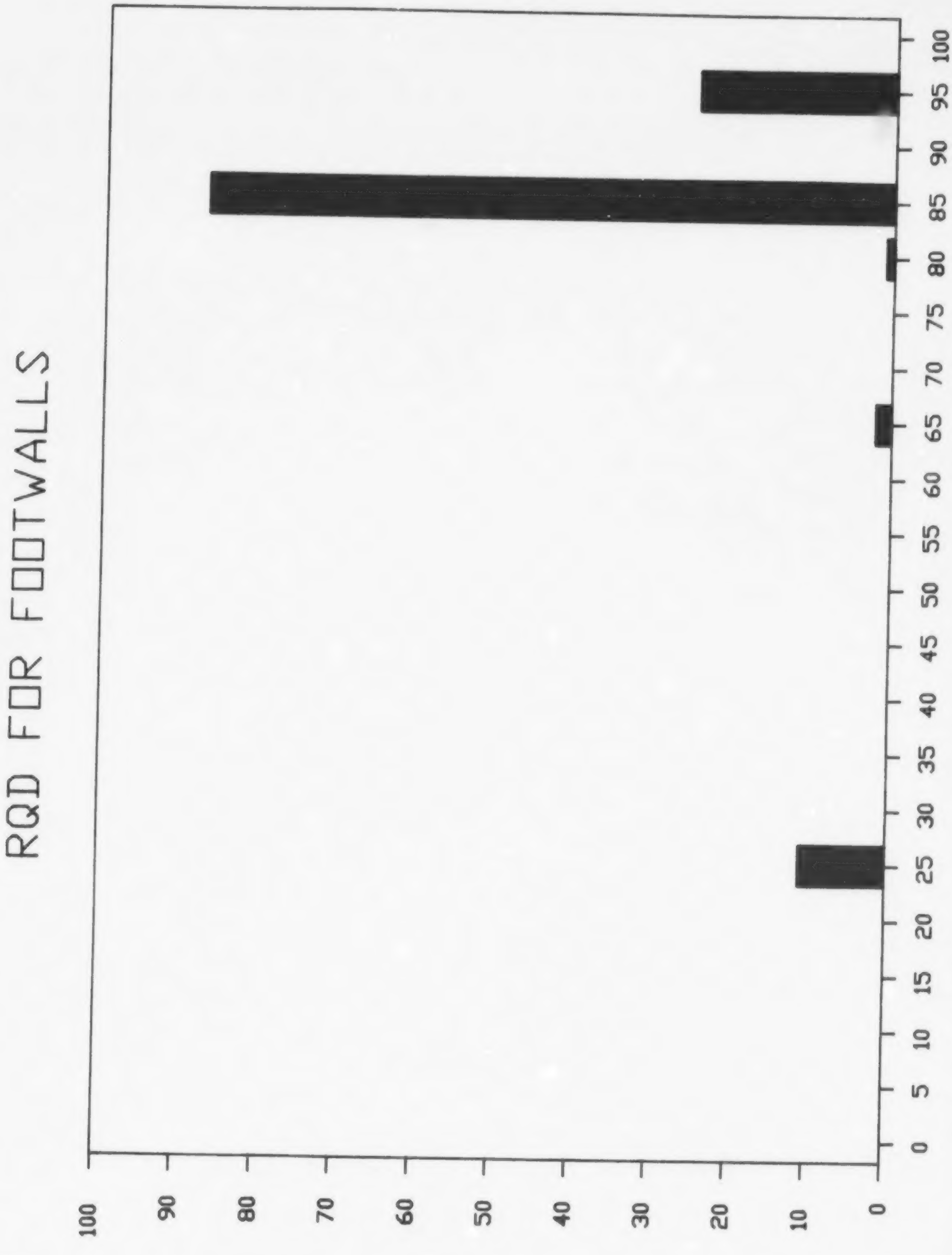
RQD — ORE ZONE

FIG. 3.29



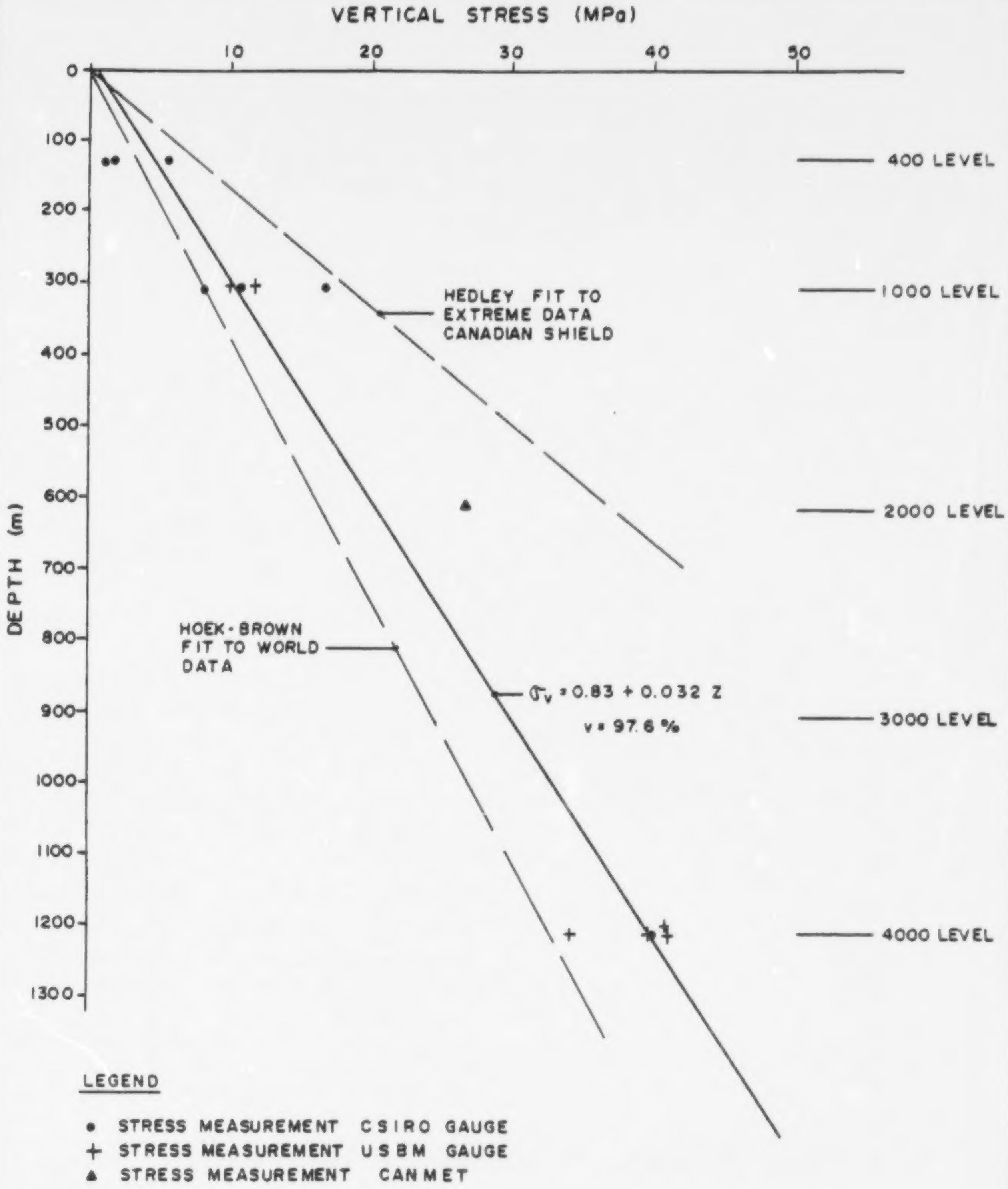
RQD FOR FOOTWALLS

FIG. 3.30



VIRGIN STRESSES

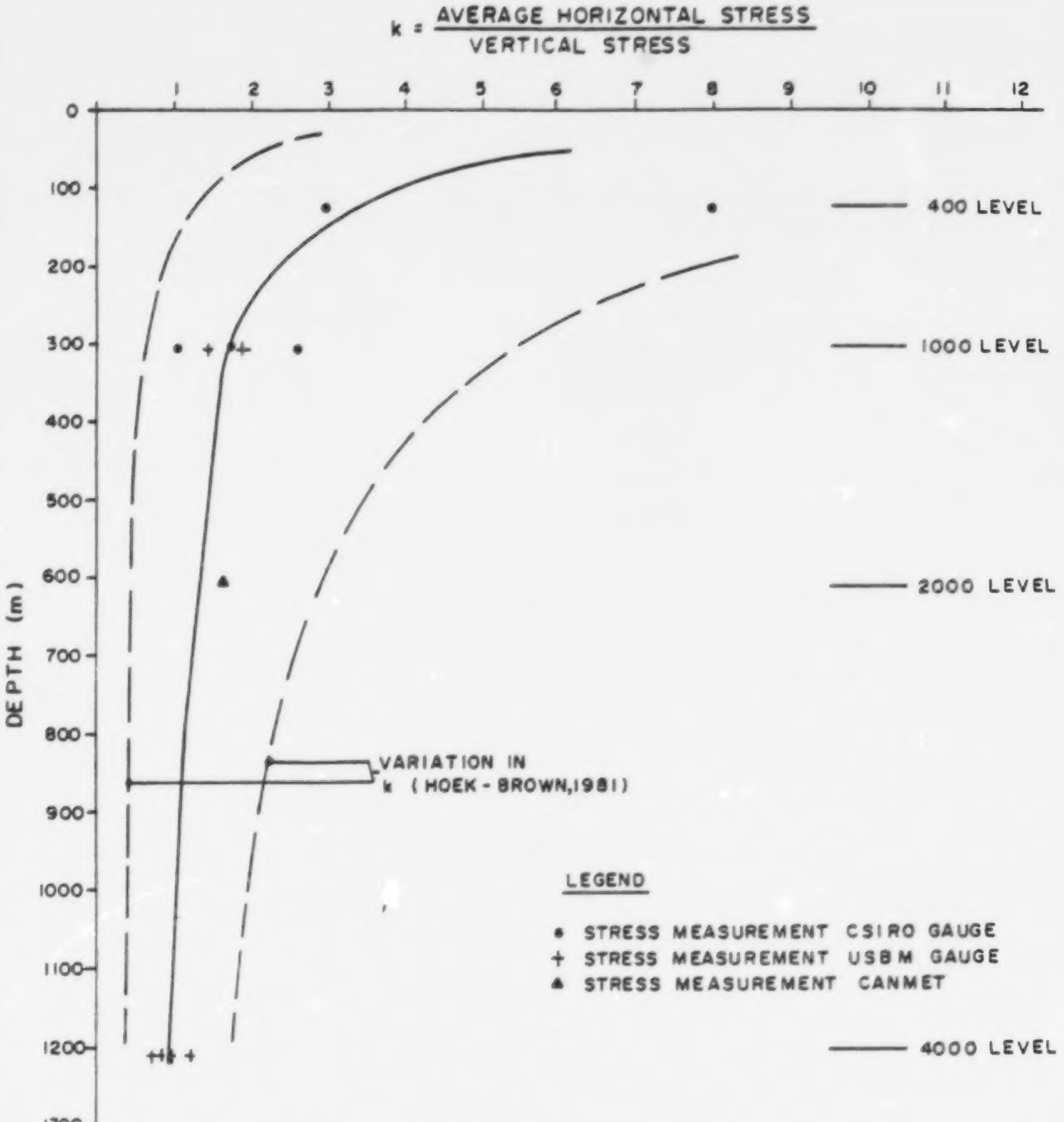
FIG. 3.31



ESTIMATED VERTICAL STRESS GRADIENT

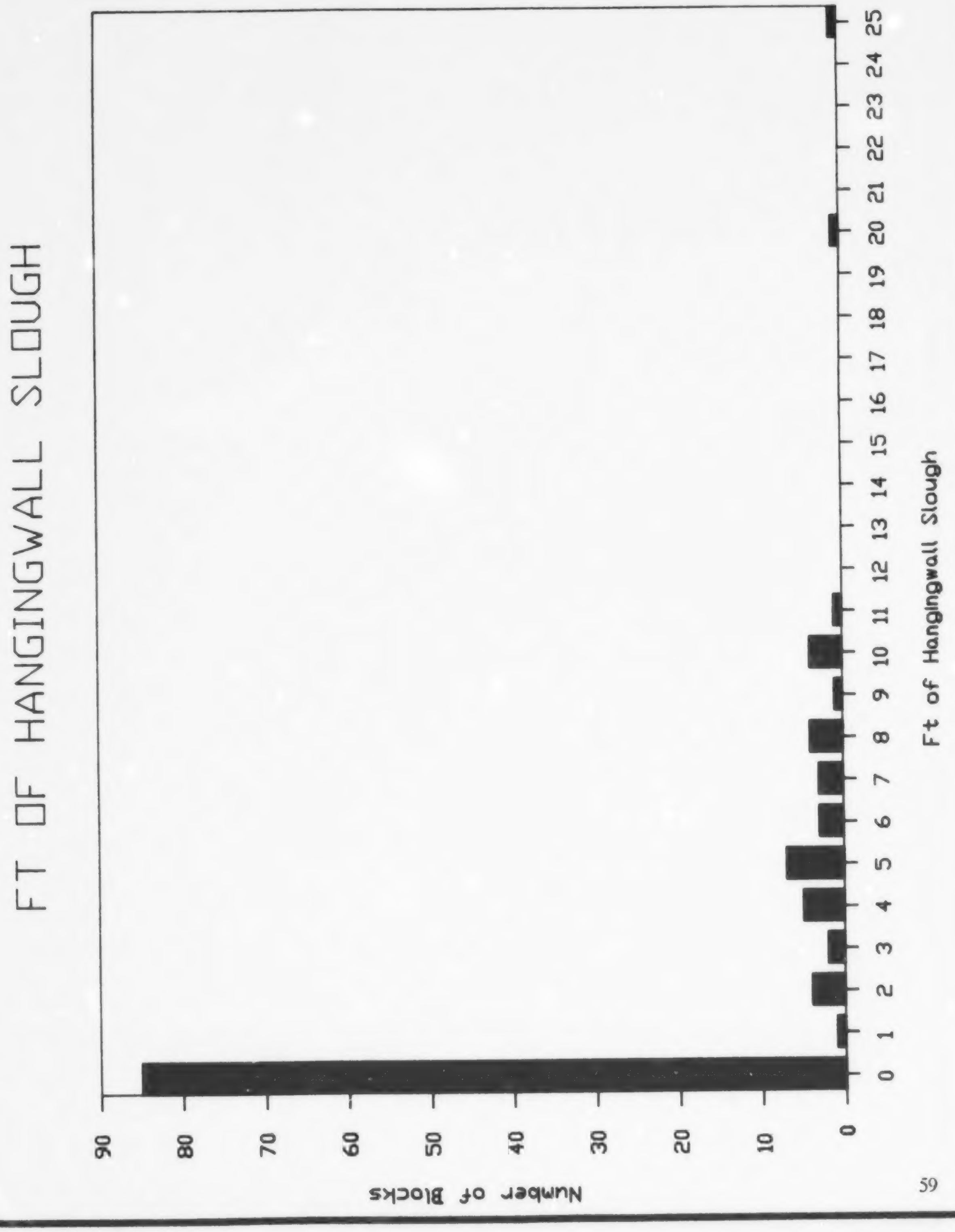
VIRGIN STRESSES

FIG. 3.32



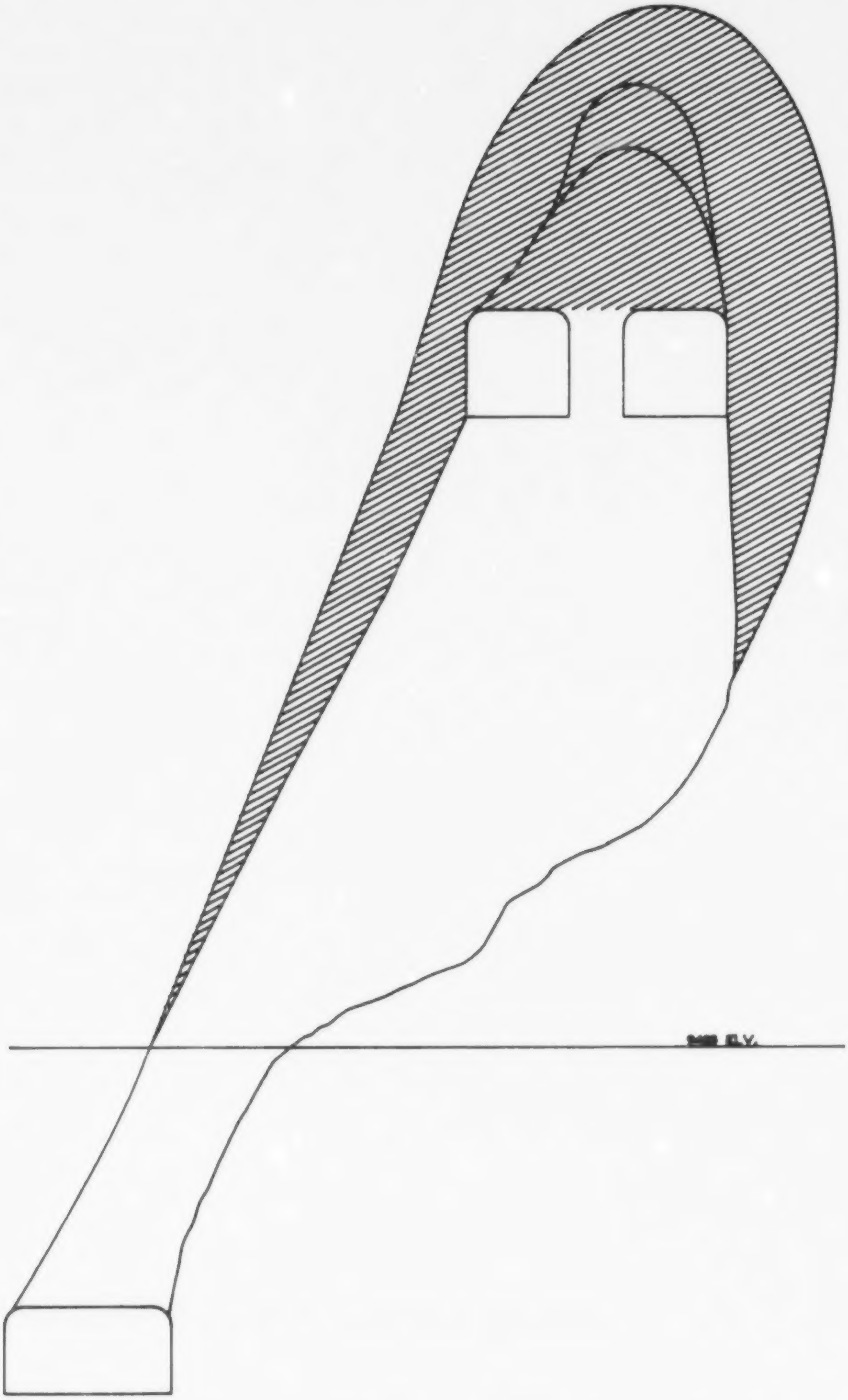
ESTIMATED K-RATIO GRADIENT

FT OF HANGINGWALL SLOUGH	FIG. 3.33



OBSERVED HANGING WALL FAILURE

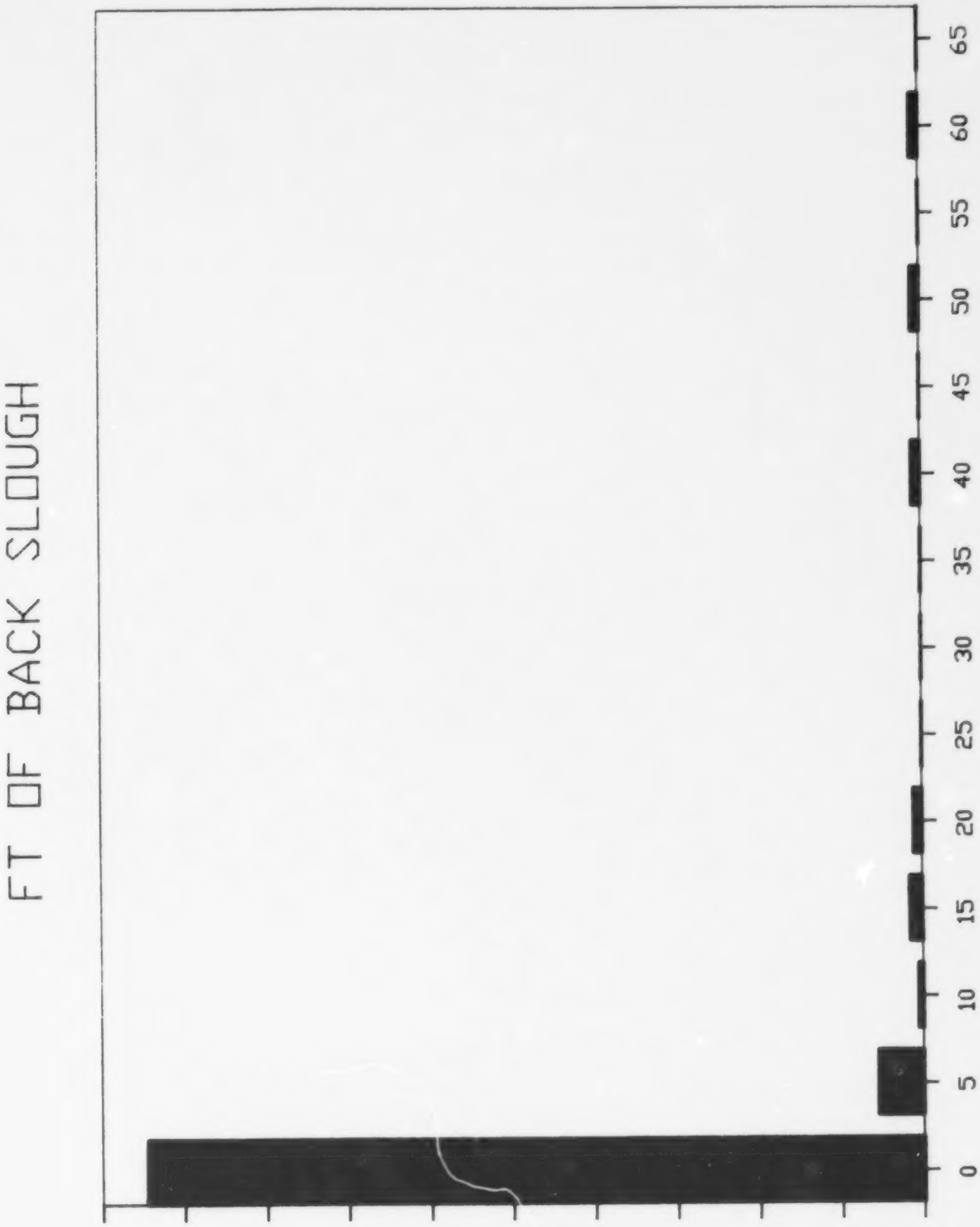
FIG. 3.34



TYPICAL SECTION (LOOKING EAST)

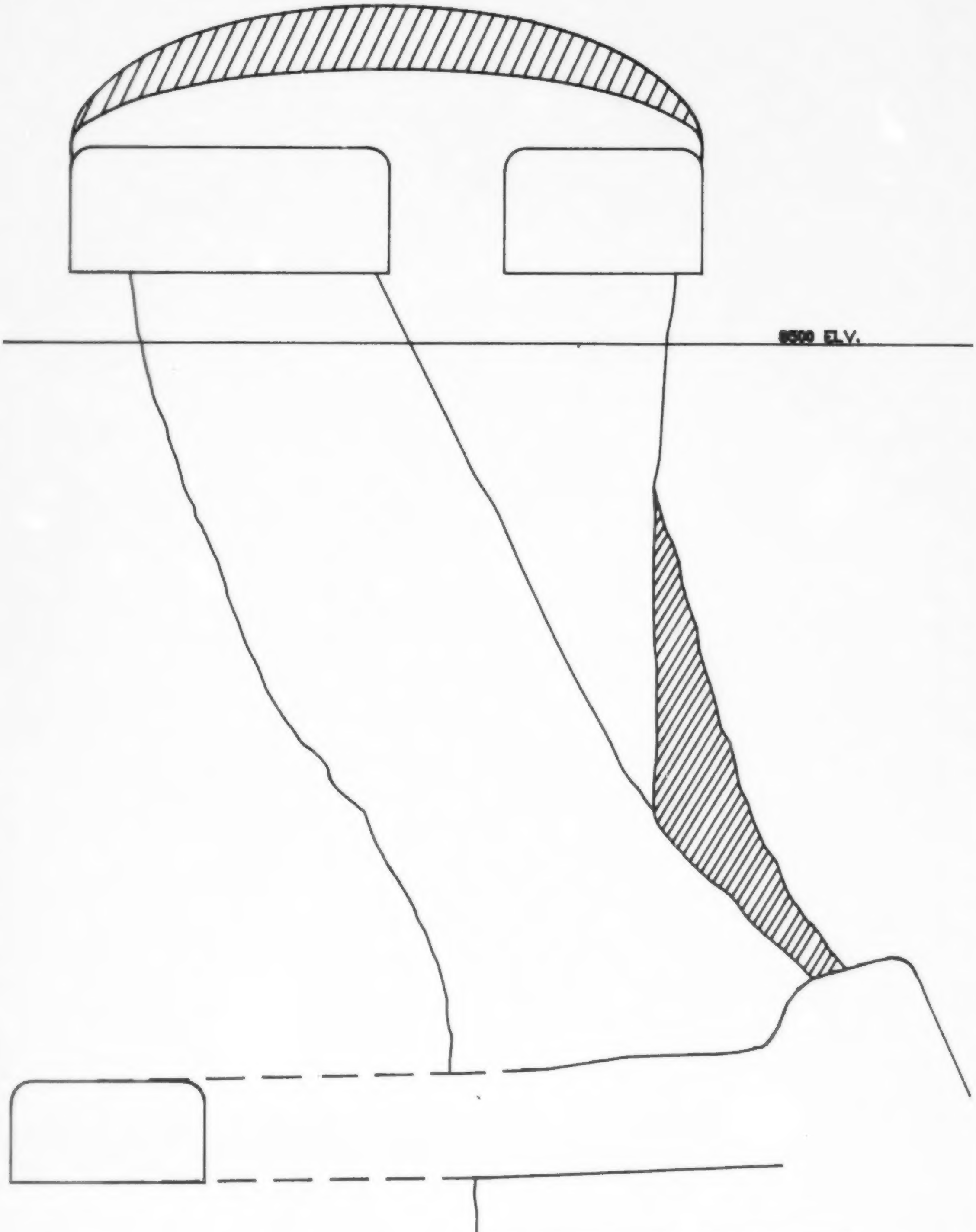
FT OF BACK SLOUGH

FIG. 3.35



OBSERVED BACK FAILURE

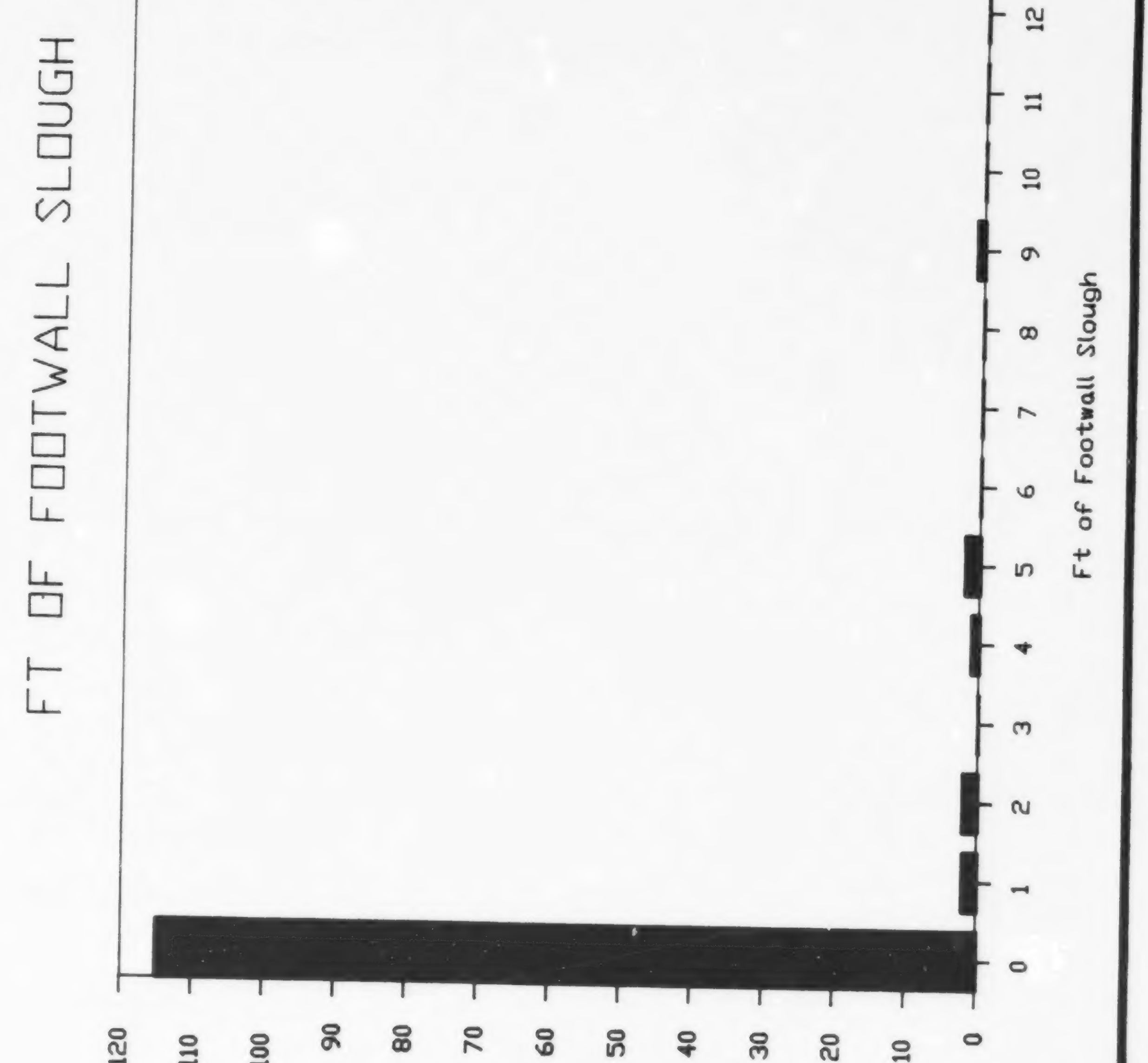
FIG. 3.36



TYPICAL SECTION (LOOKING EAST)

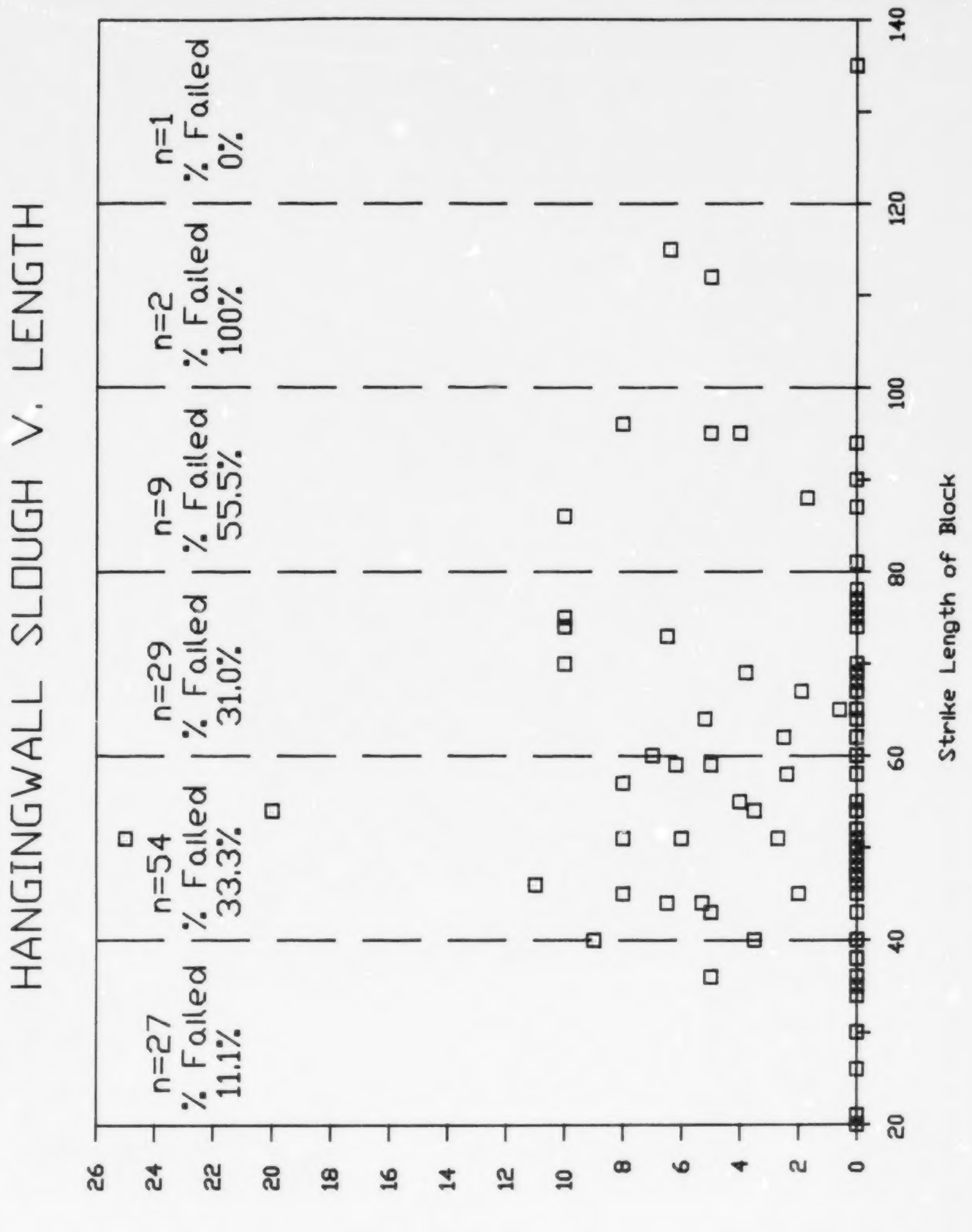
FT OF FOOTWALL SLOUGH

FIG. 3.37



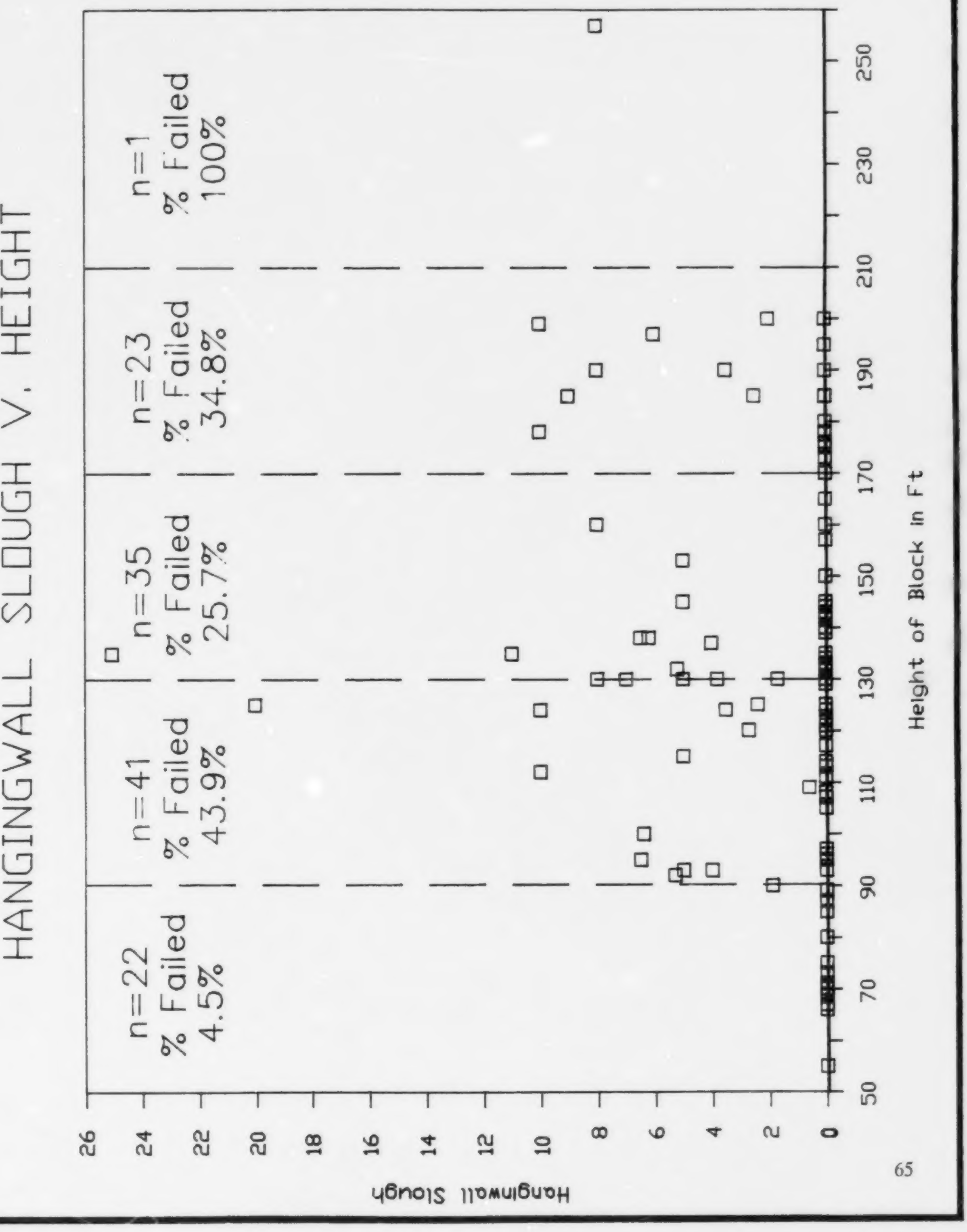
HANGINGWALL SLOUGH V. LENGTH

FIG. 3.38



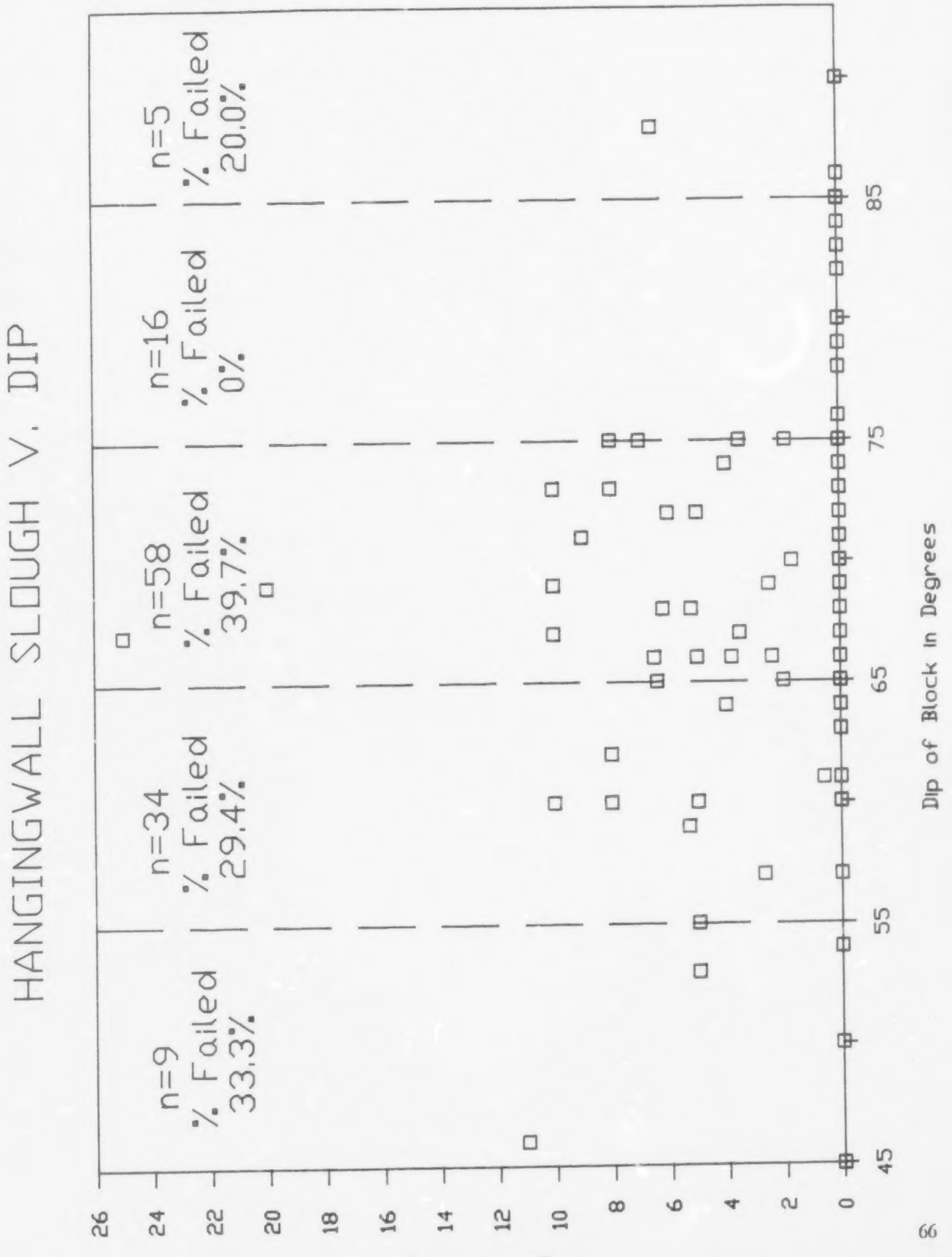
HANGINGWALL SLOUGH V. HEIGHT

FIG. 3.39



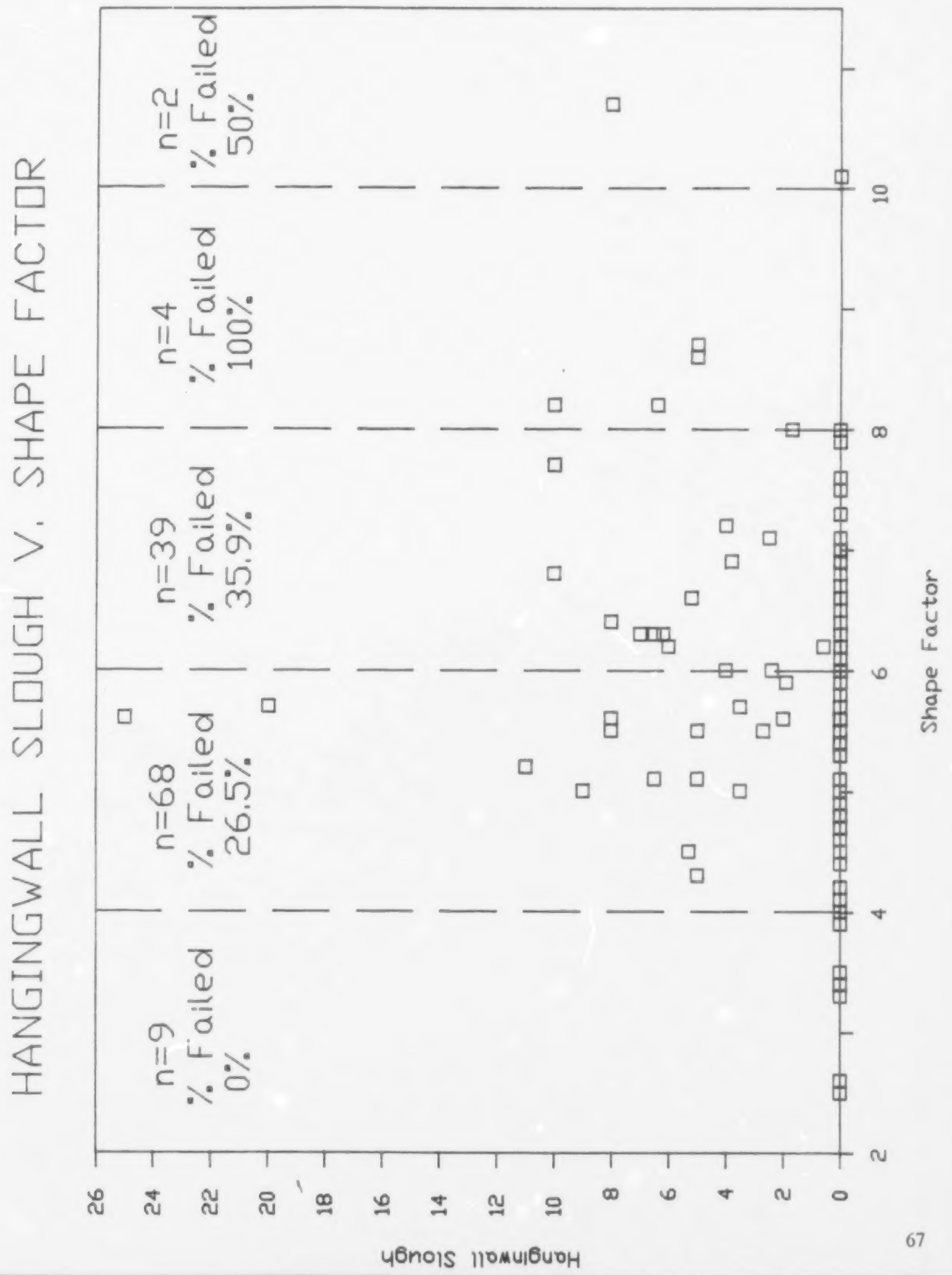
HANGINGWALL SLOUGH V. DIP

FIG. 3.40



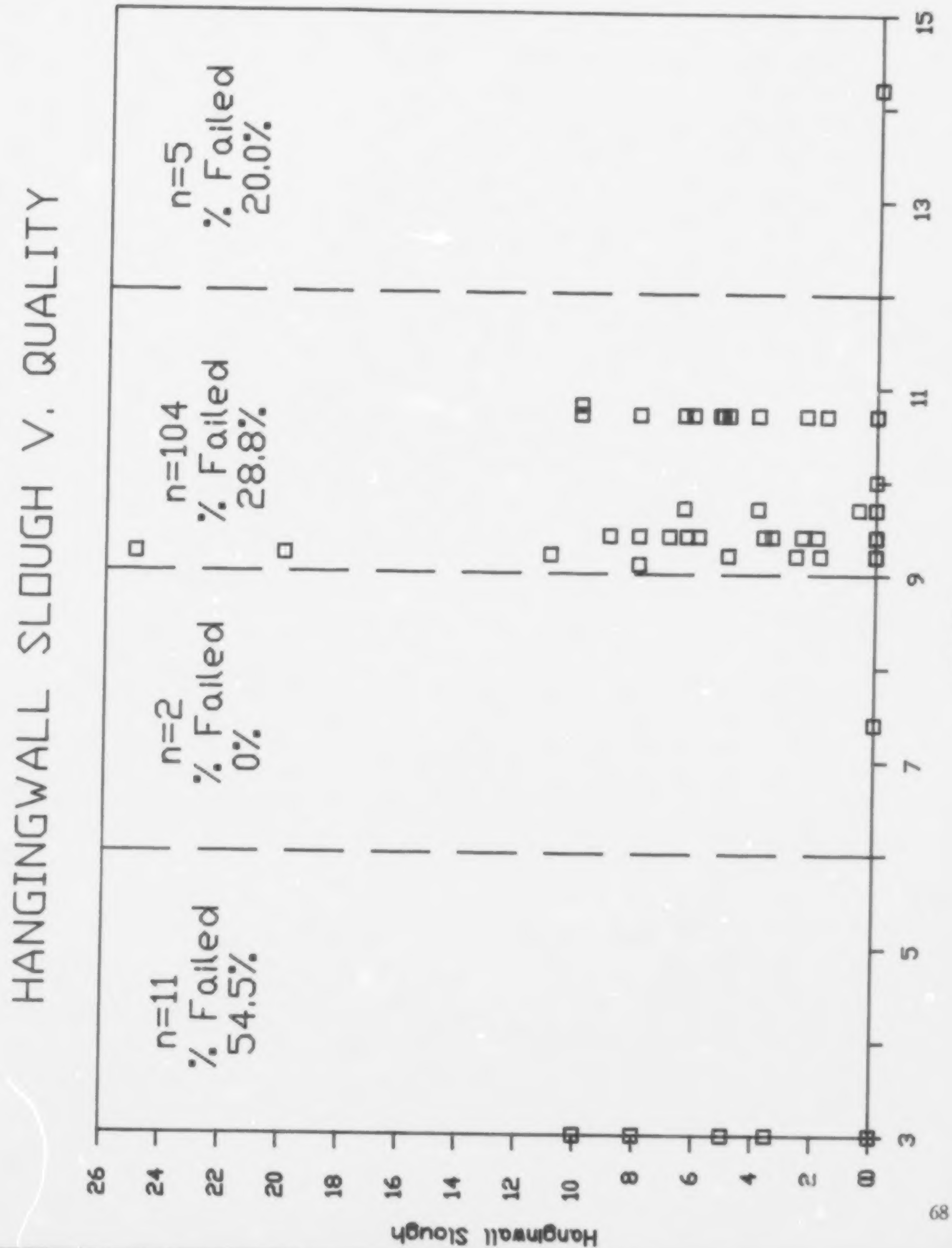
HANGINGWALL SLOUGH V. SHAPE FACTOR

FIG. 3.41



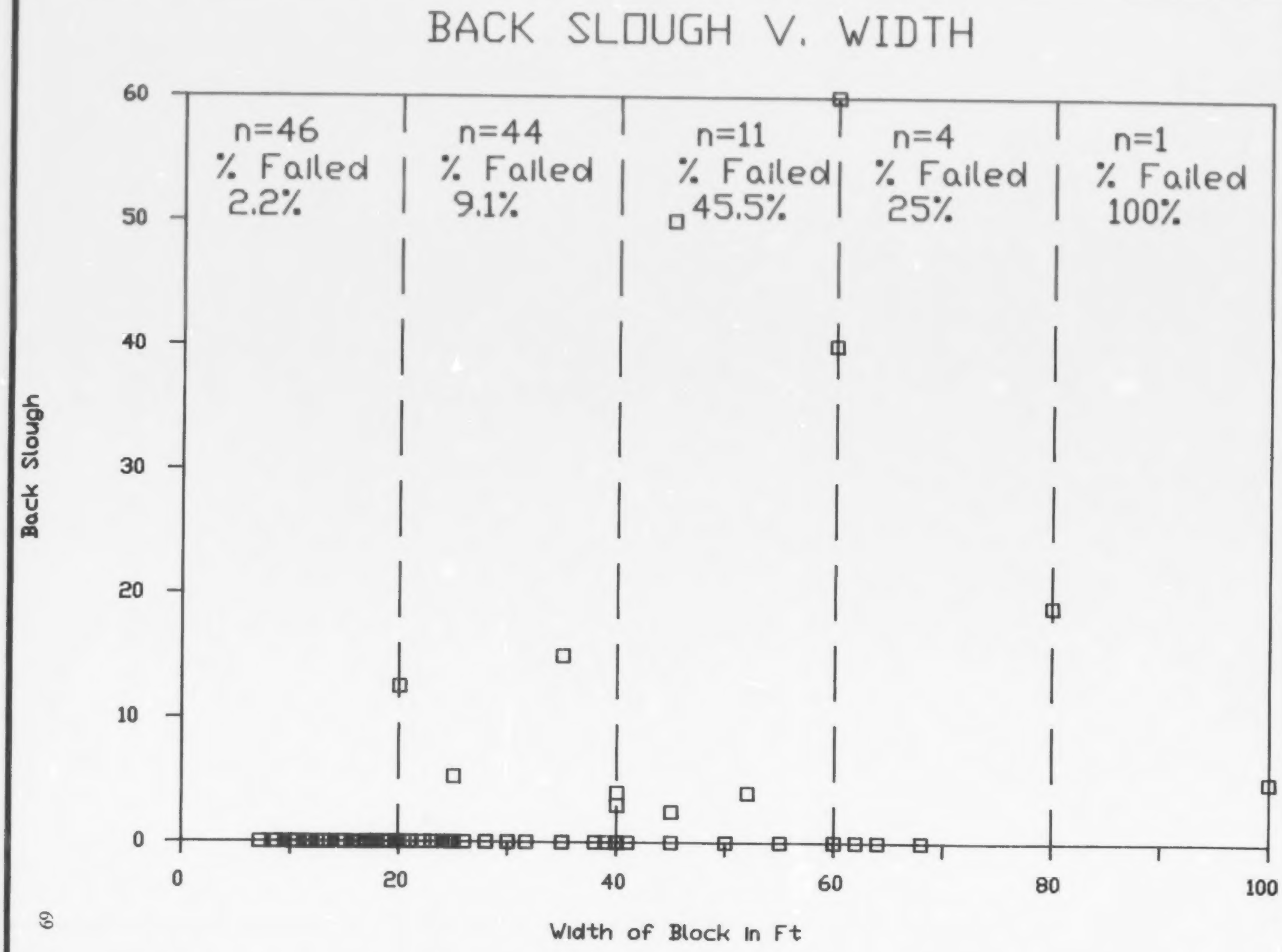
HANGINGWALL SLOUGH V. QUALITY

FIG. 3.42



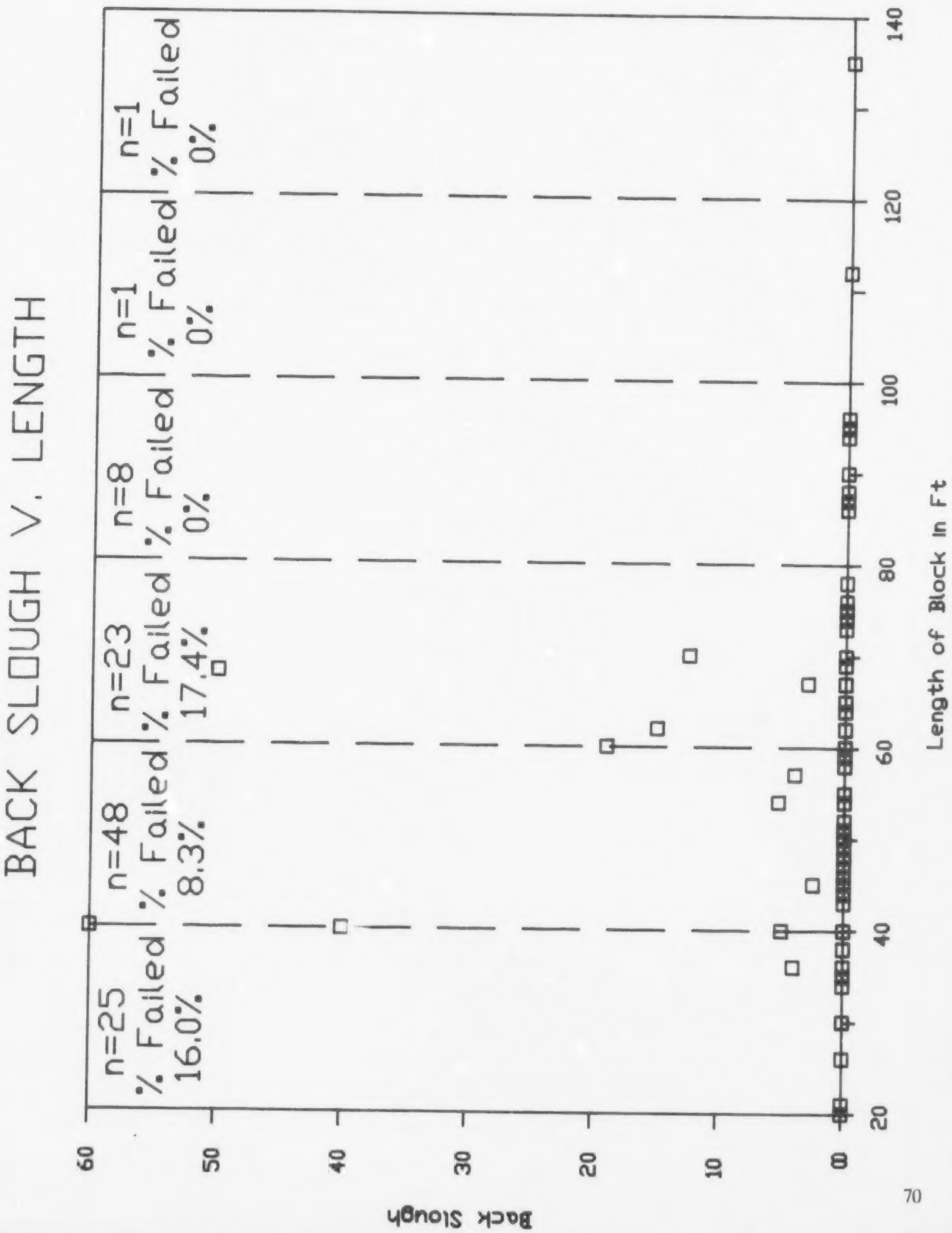
BACK SLOUGH V. WIDTH

FIG. 3.43



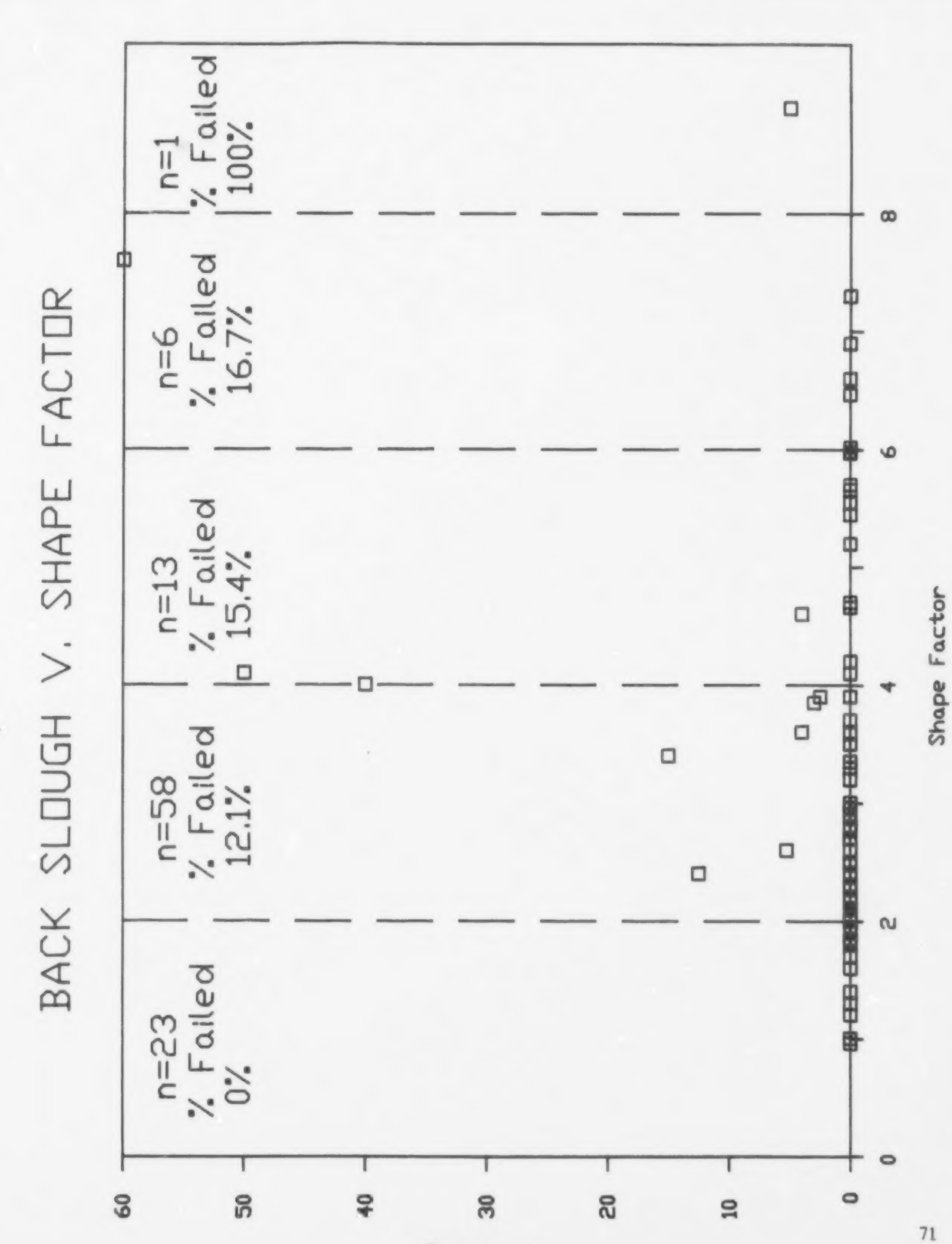
BACK SLOUGH V. LENGTH

FIG. 3.44



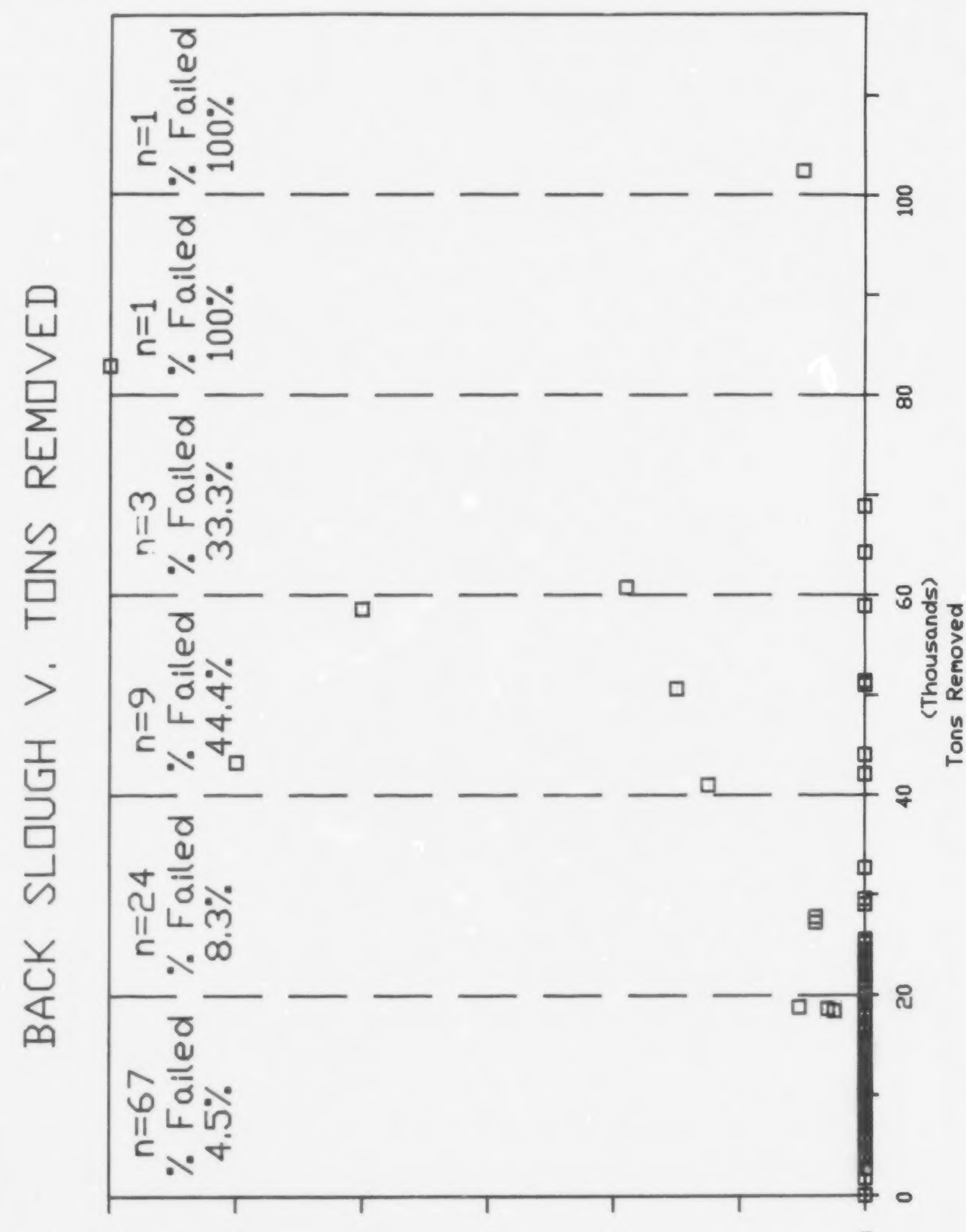
BACK SLOUGH V. SHAPE FACTOR

FIG. 3.45



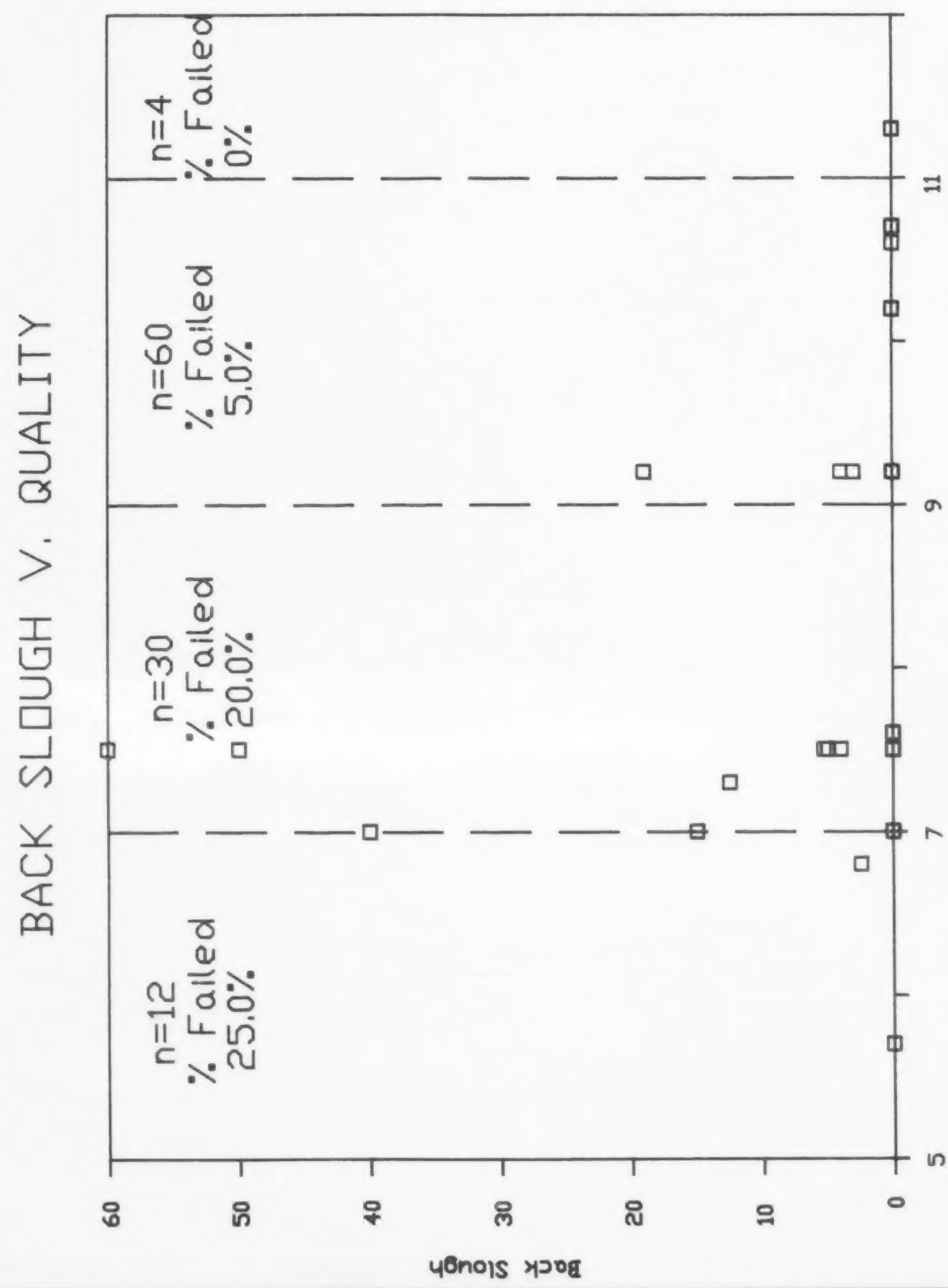
BACK SLOUGH V. TONS REMOVED

FIG. 3.46



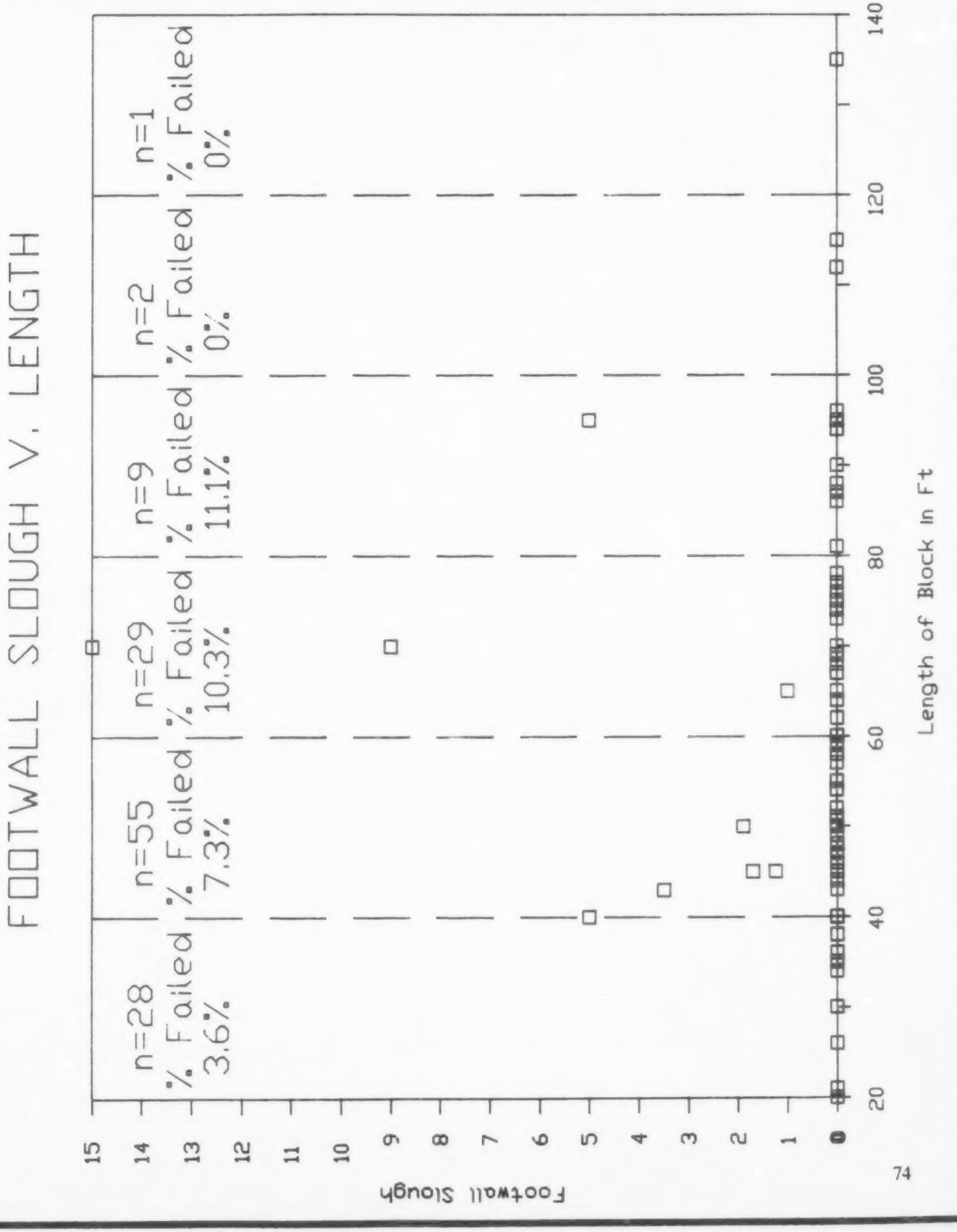
BACK SLOUGH V. QUALITY

FIG. 3.47



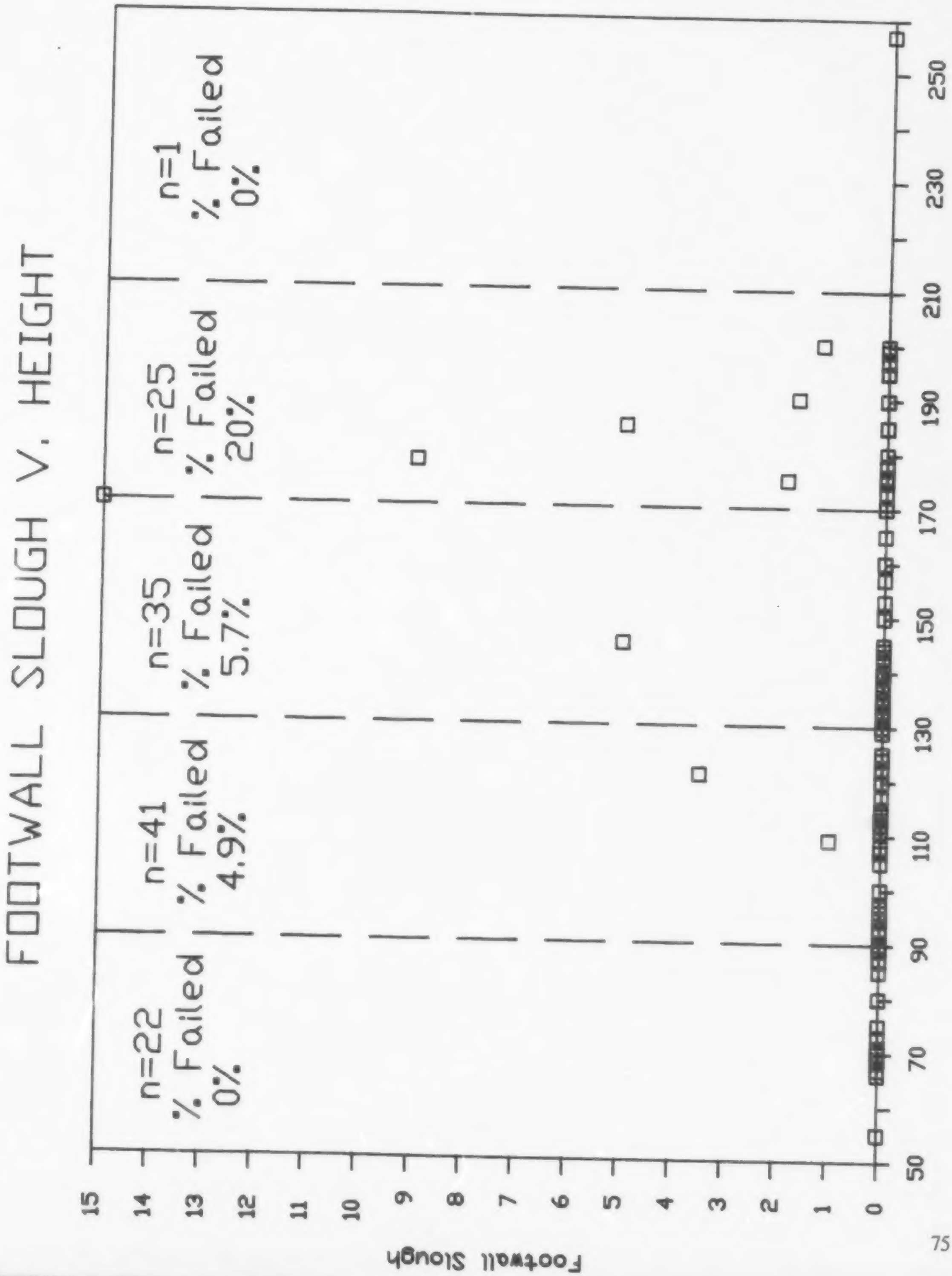
FOOTWALL SLOUGH V. LENGTH

FIG. 3.48



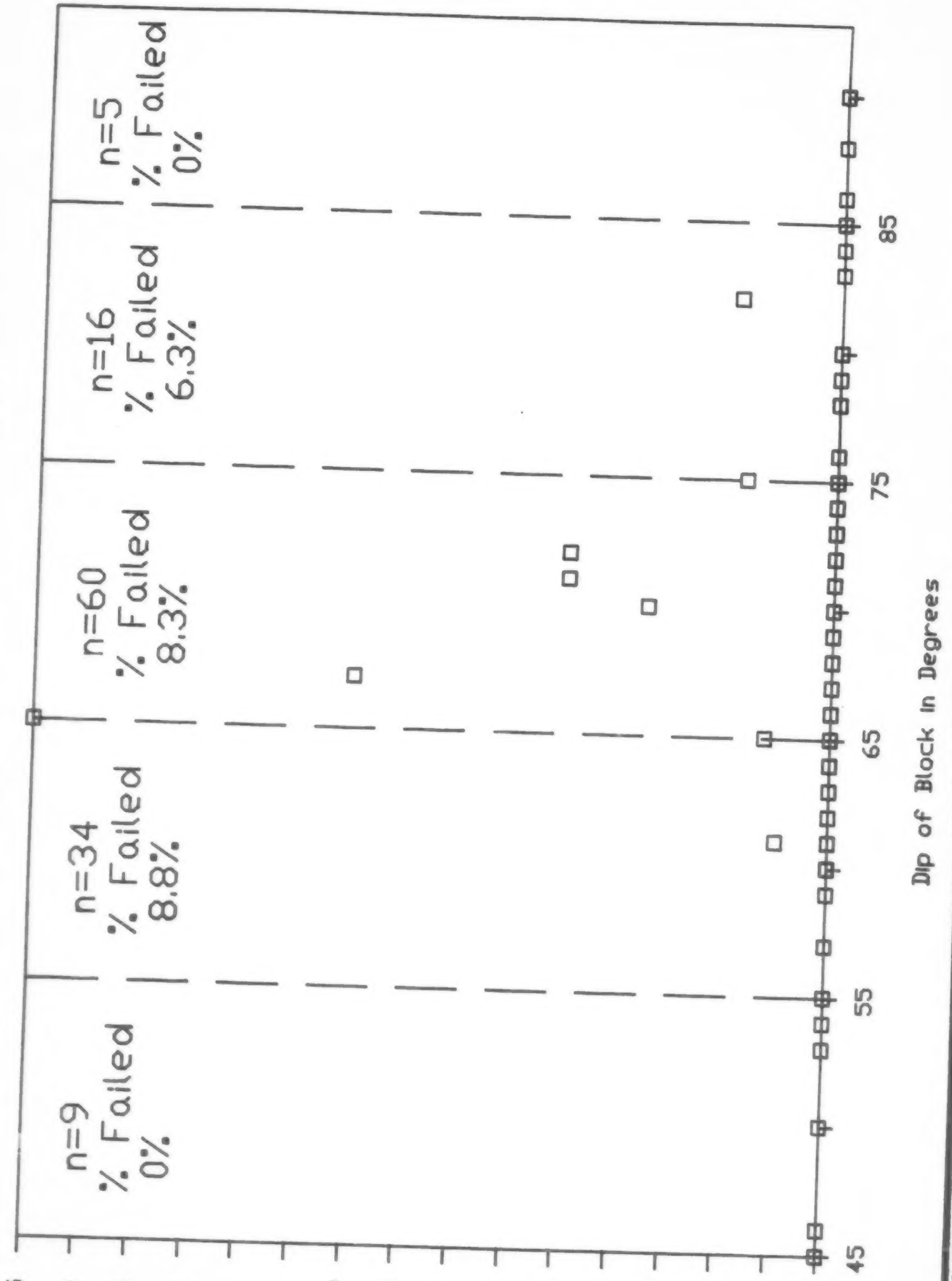
FOOTWALL SLOUGH V. HEIGHT

FIG. 3.49



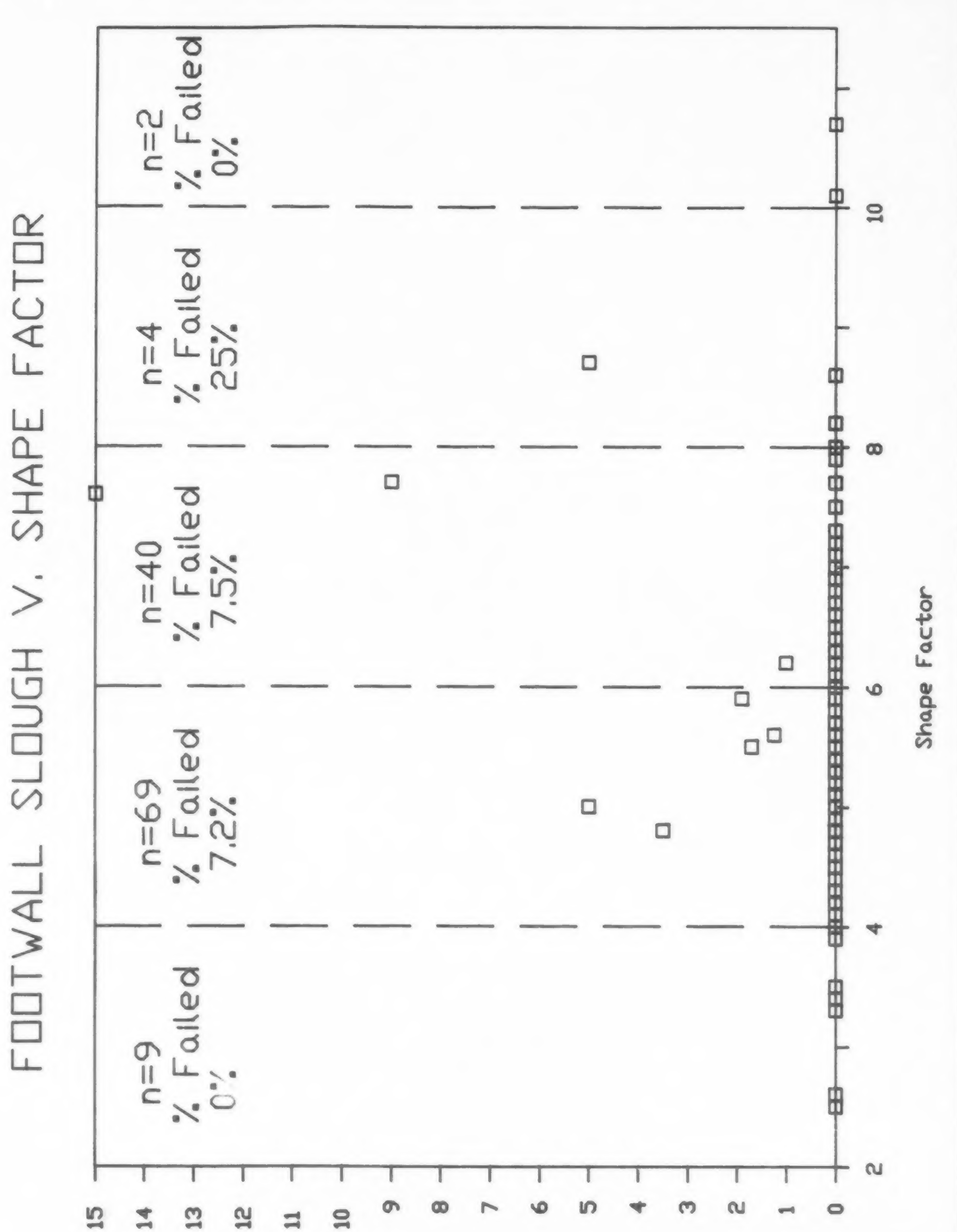
FOOTWALL SLOUGH V. DIP

FIG. 3.50



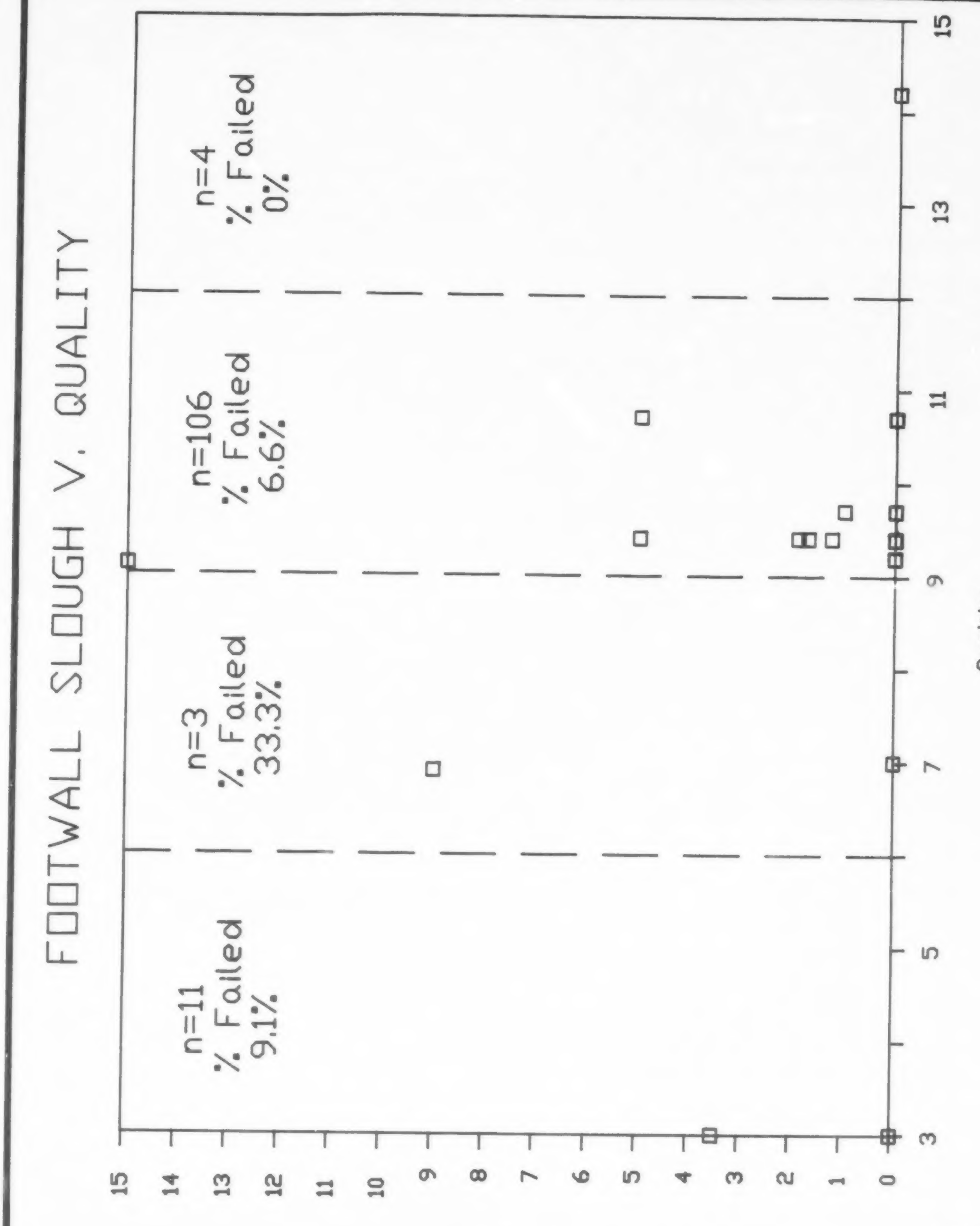
FOOTWALL SLOUGH V. SHAPE FACTOR

FIG. 3.51



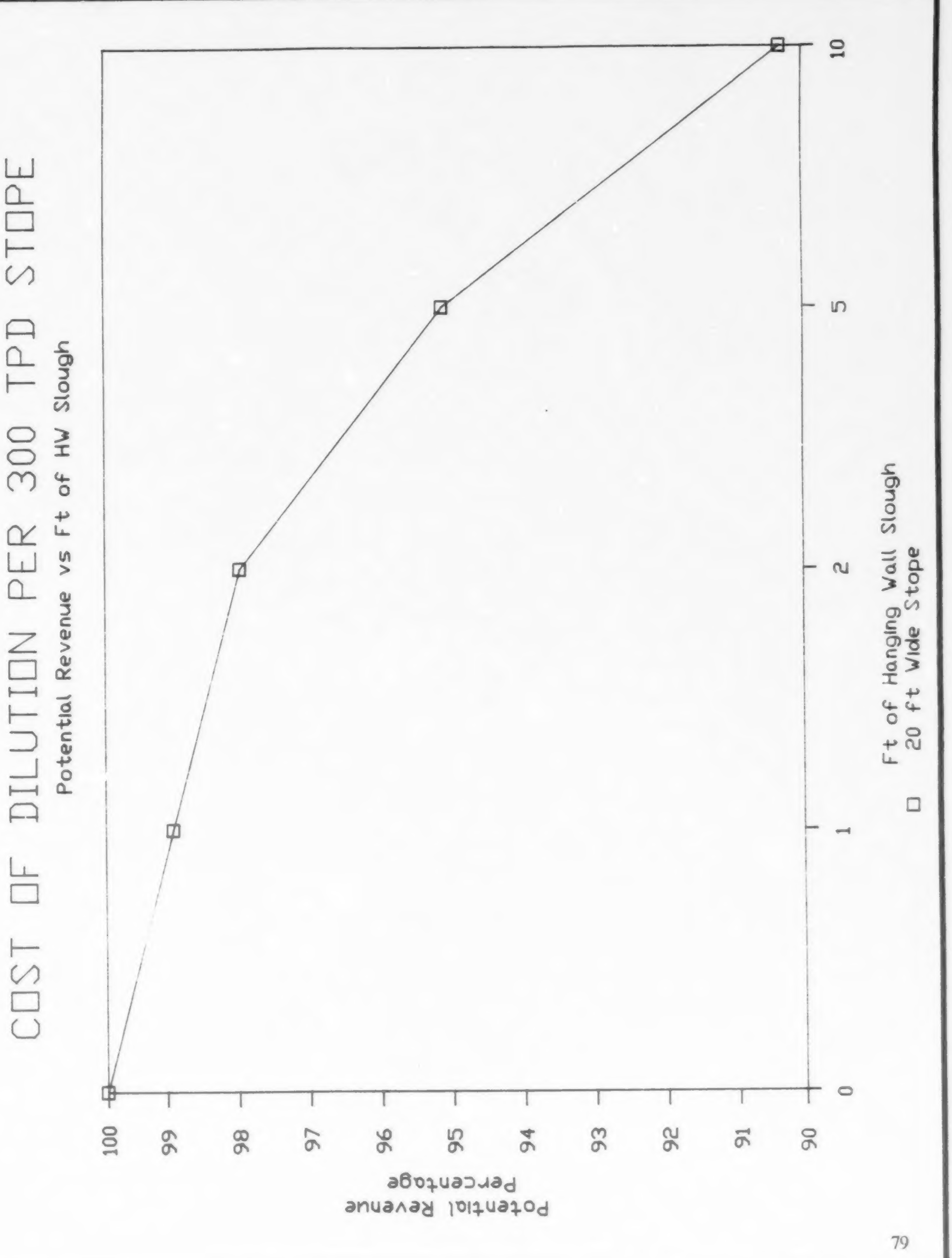
FOOTWALL SLOUGH V. QUALITY

FIG. 3.52



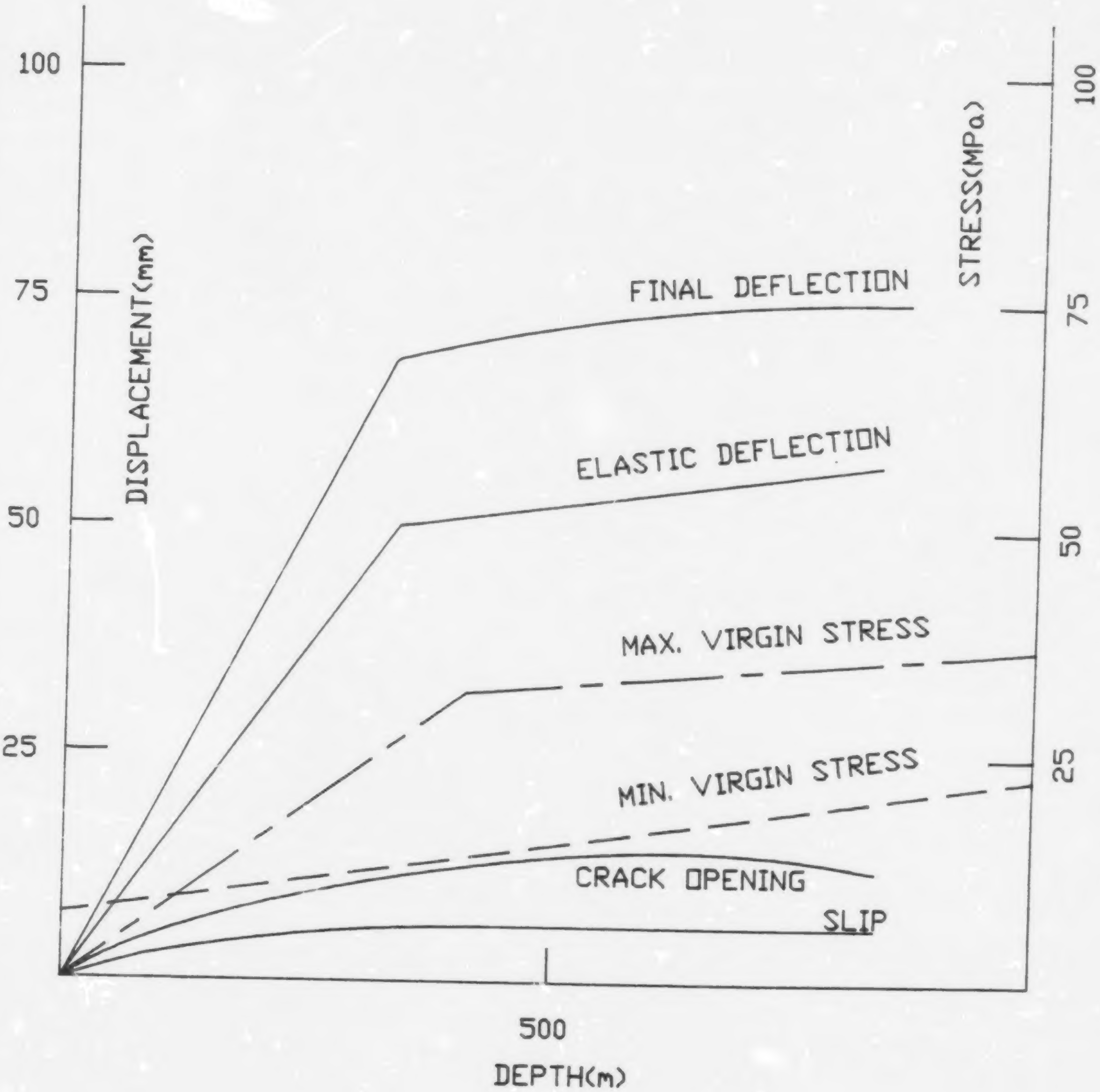
COST OF DILUTION PER 300 TPD STOPE

FIG. 3.53



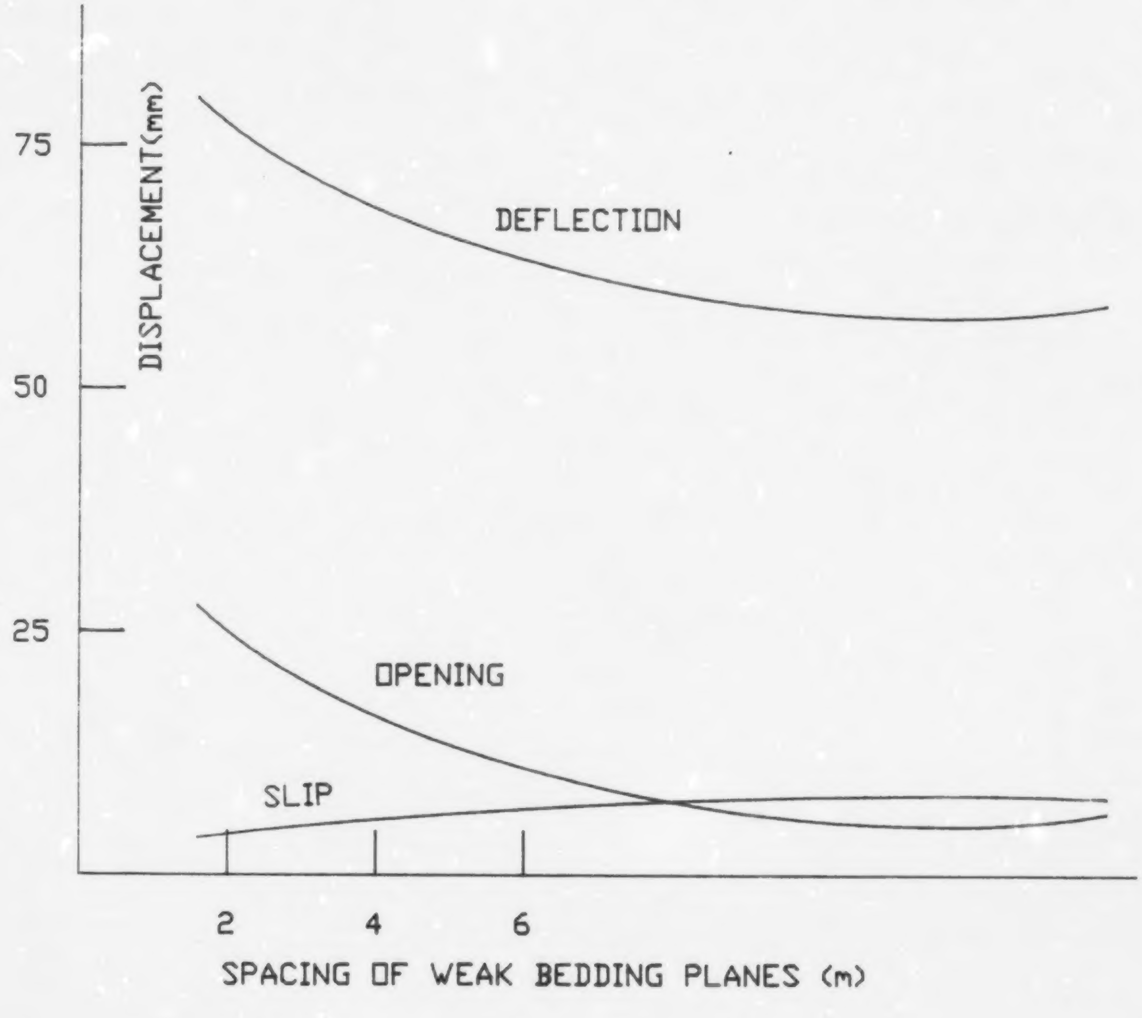
INFLUENCE OF DEPTH BELOW SURFACE

FIG. 3.54



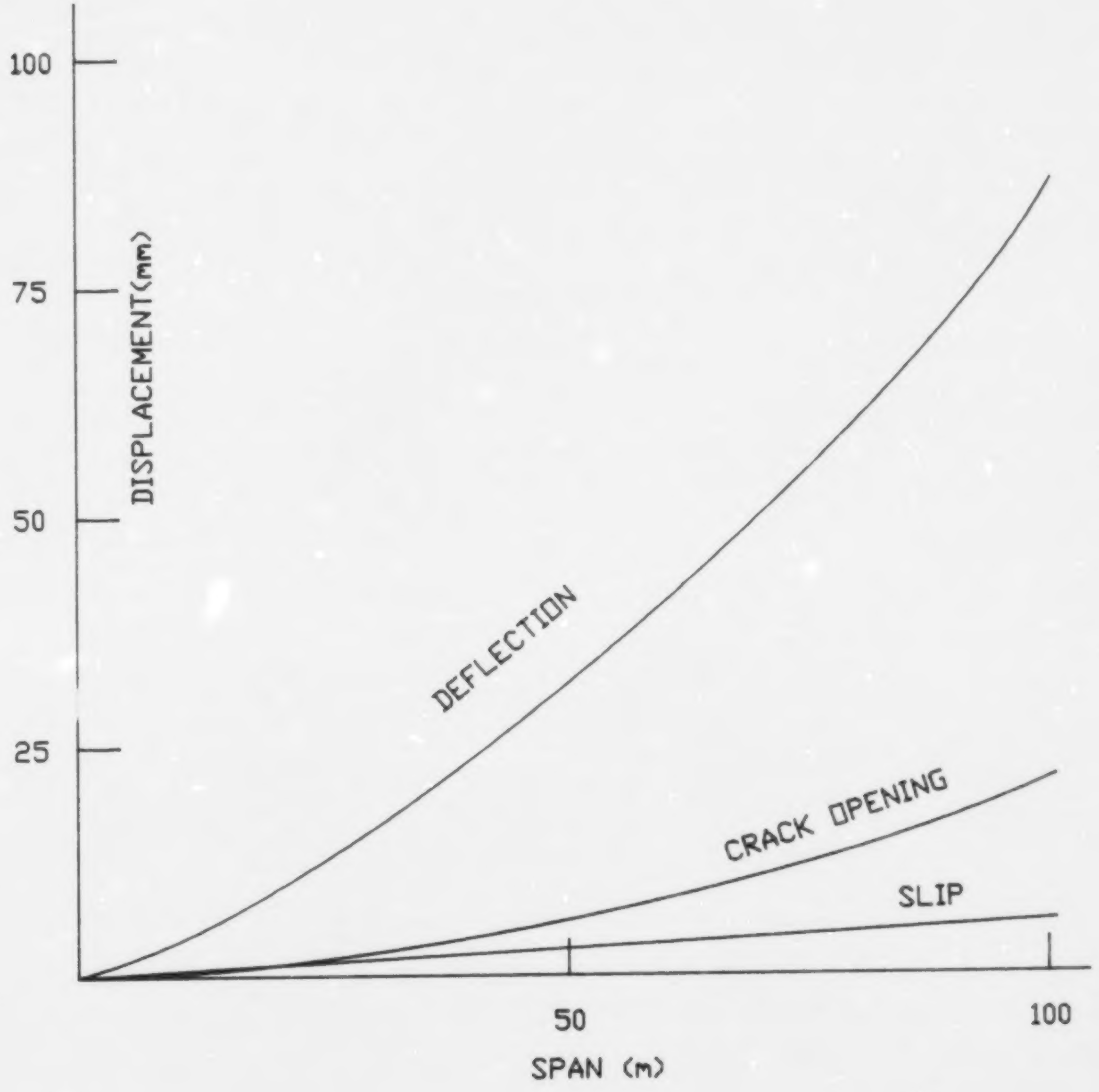
INFLUENCE OF FOLIATION SPACING

FIG. 3.55



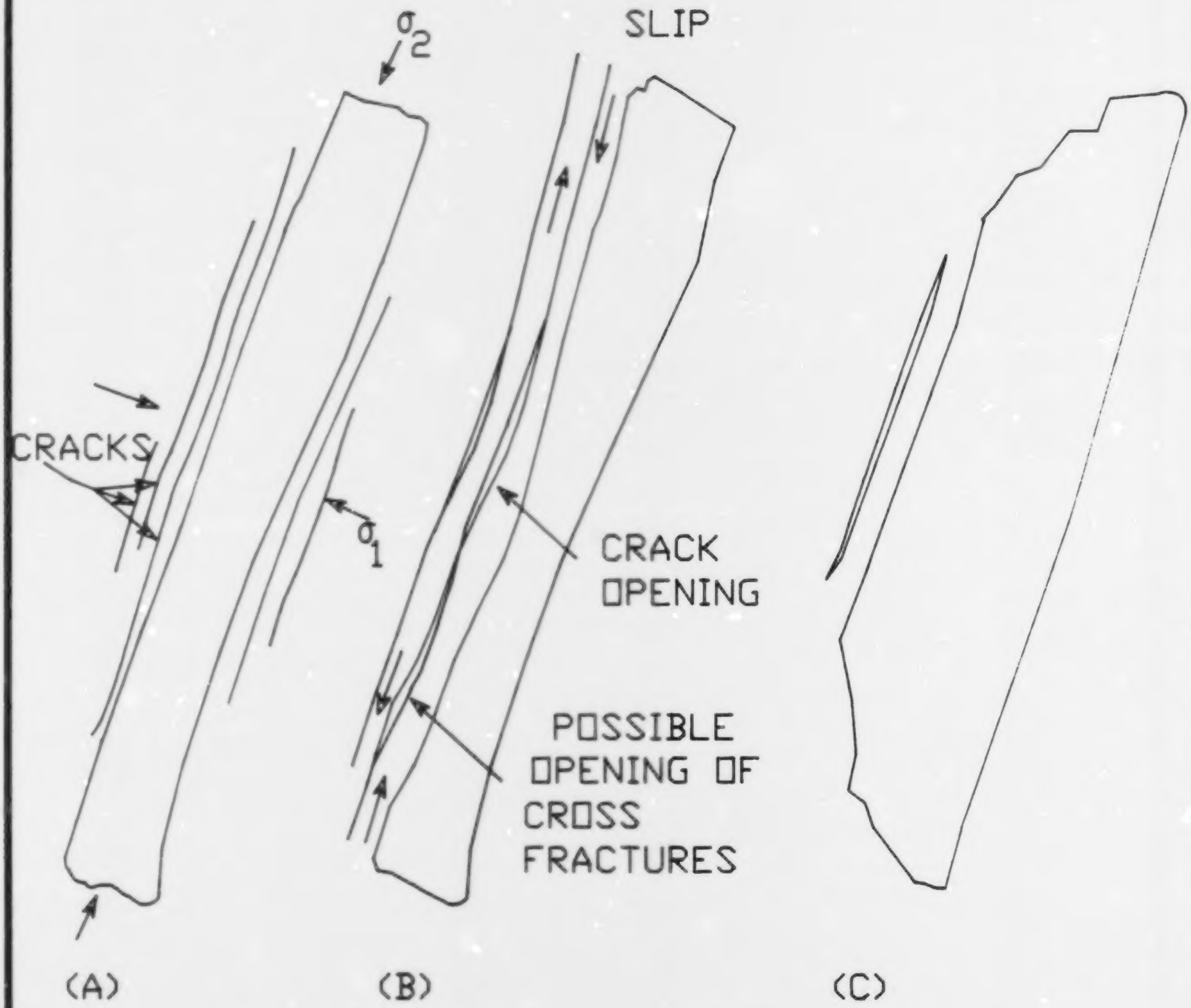
INFLUENCE OF SPAN

FIG. 3.56



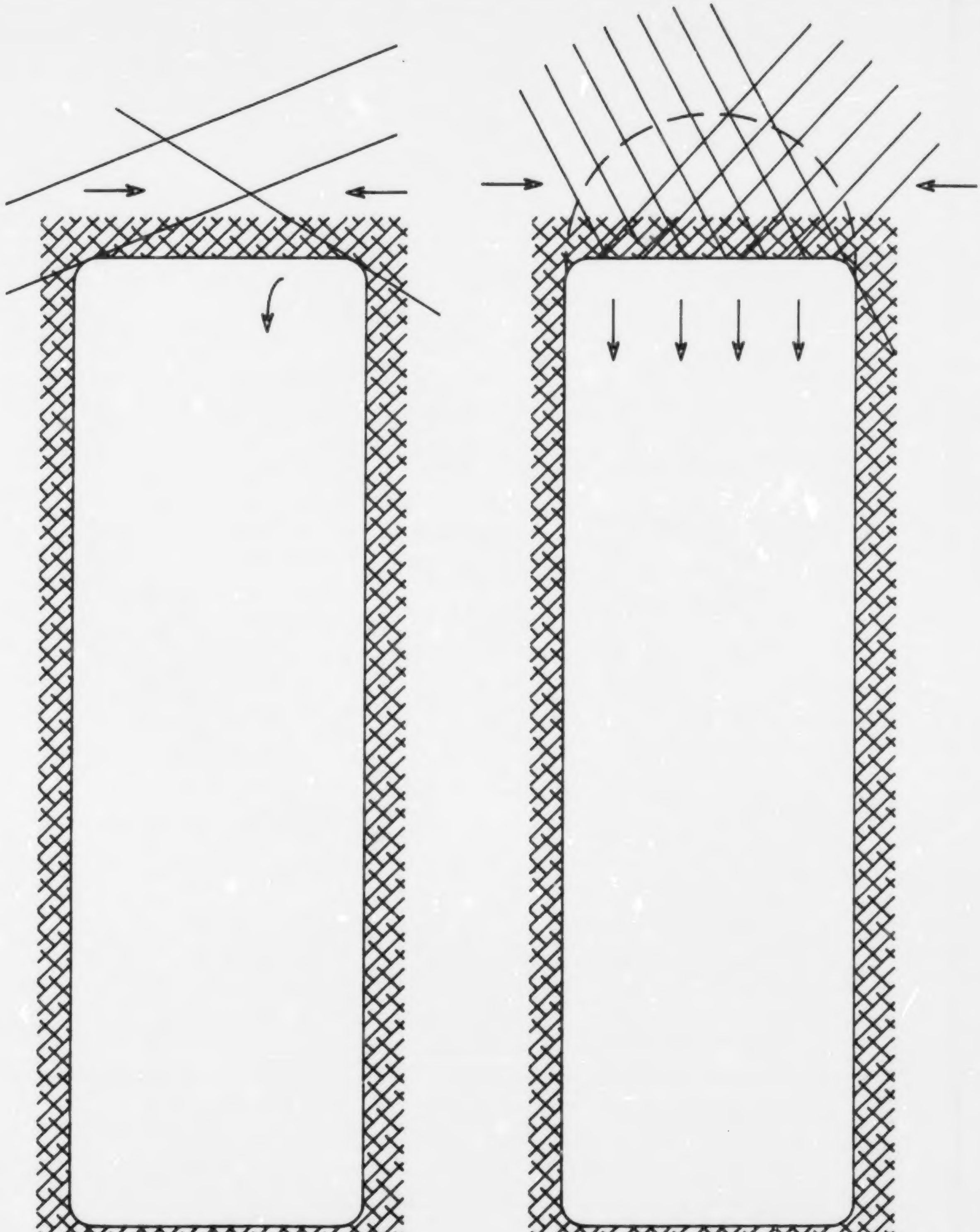
MECHANISM OF HANGING WALL FAILURE

FIG. 3.57


σ_2
SLIP
CRACKS
σ_1
CRACK OPENING
POSSIBLE OPENING OF CROSS FRACTURES
(A)
(B)
(C)

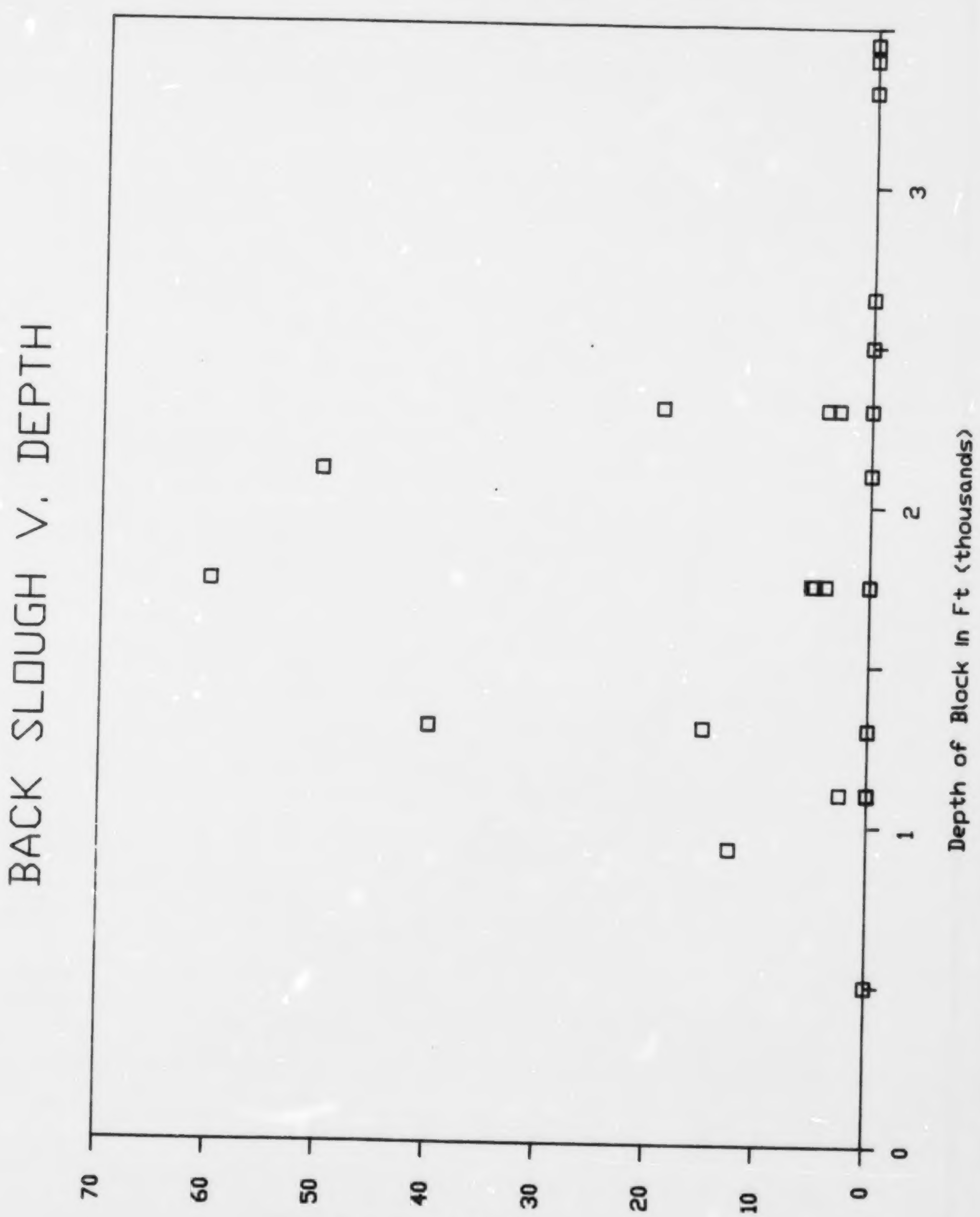
TYPICAL BACK FAILURES

FIG. 3.58



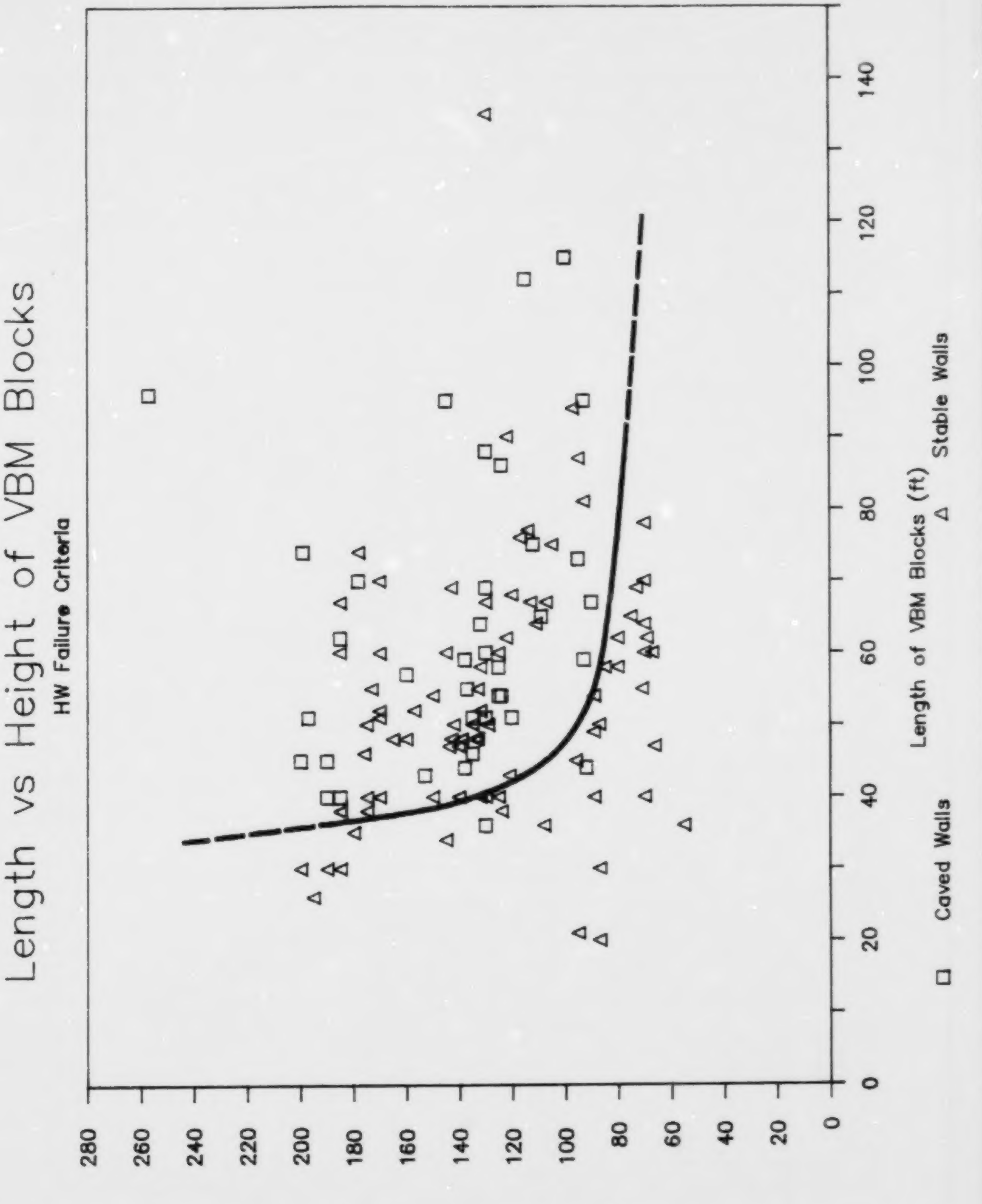
BACK SLOUGH V. DEPTH

FIG. 3.59



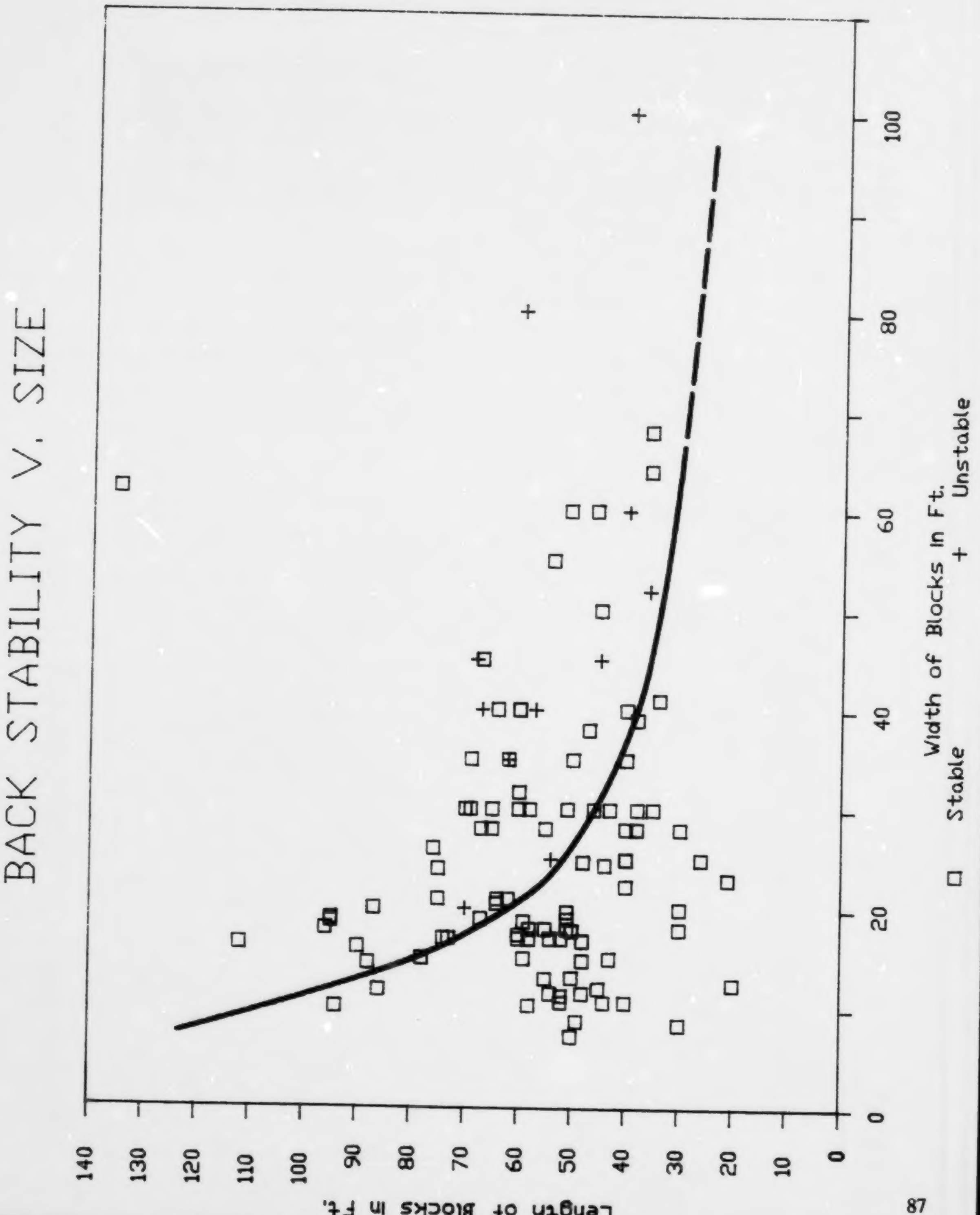
HANGINGWALL STABILITY V. SIZE AT THOMPSON MINE

FIG. 3.60



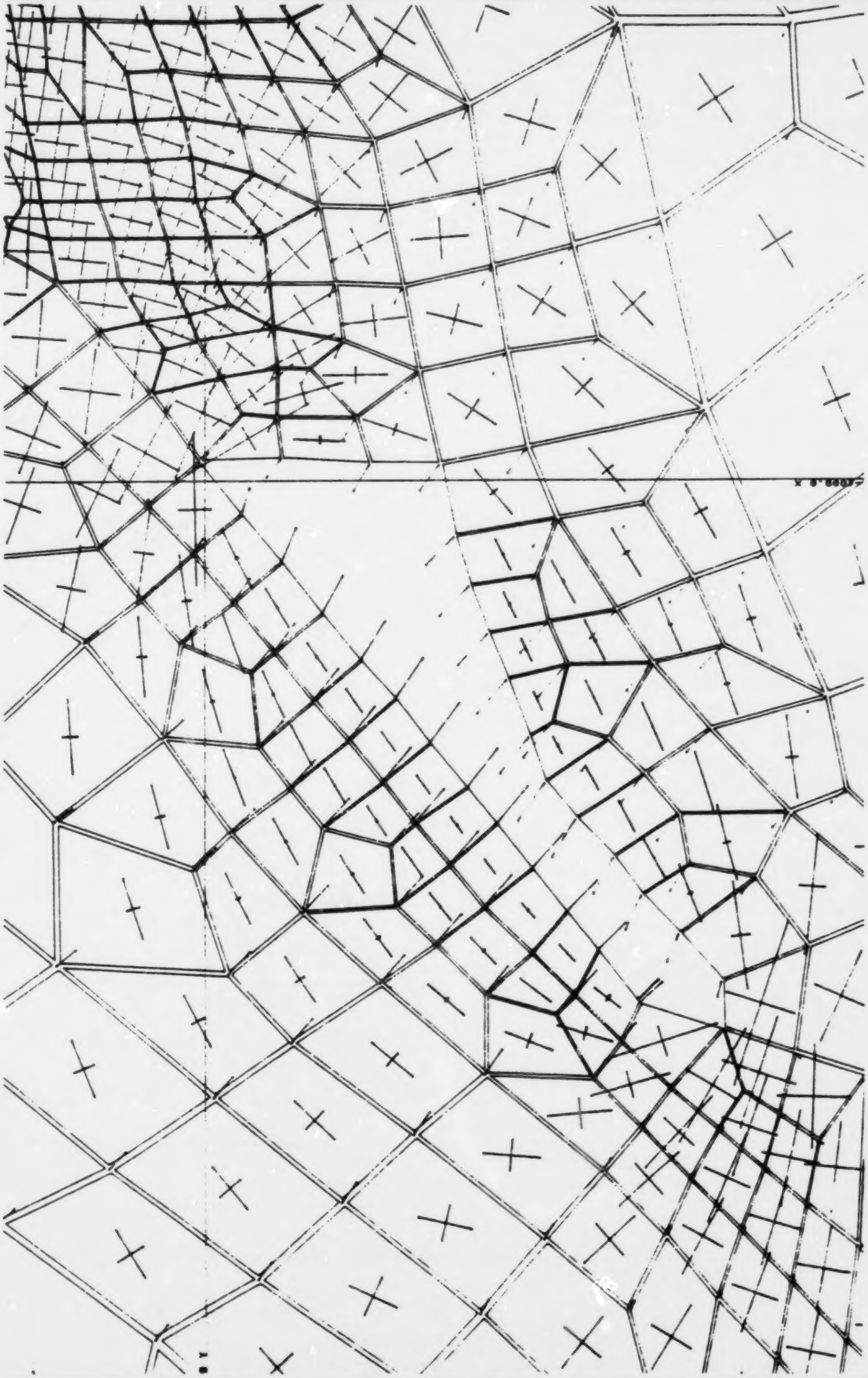
BACK STABILITY V. SIZE

FIG. 3.61



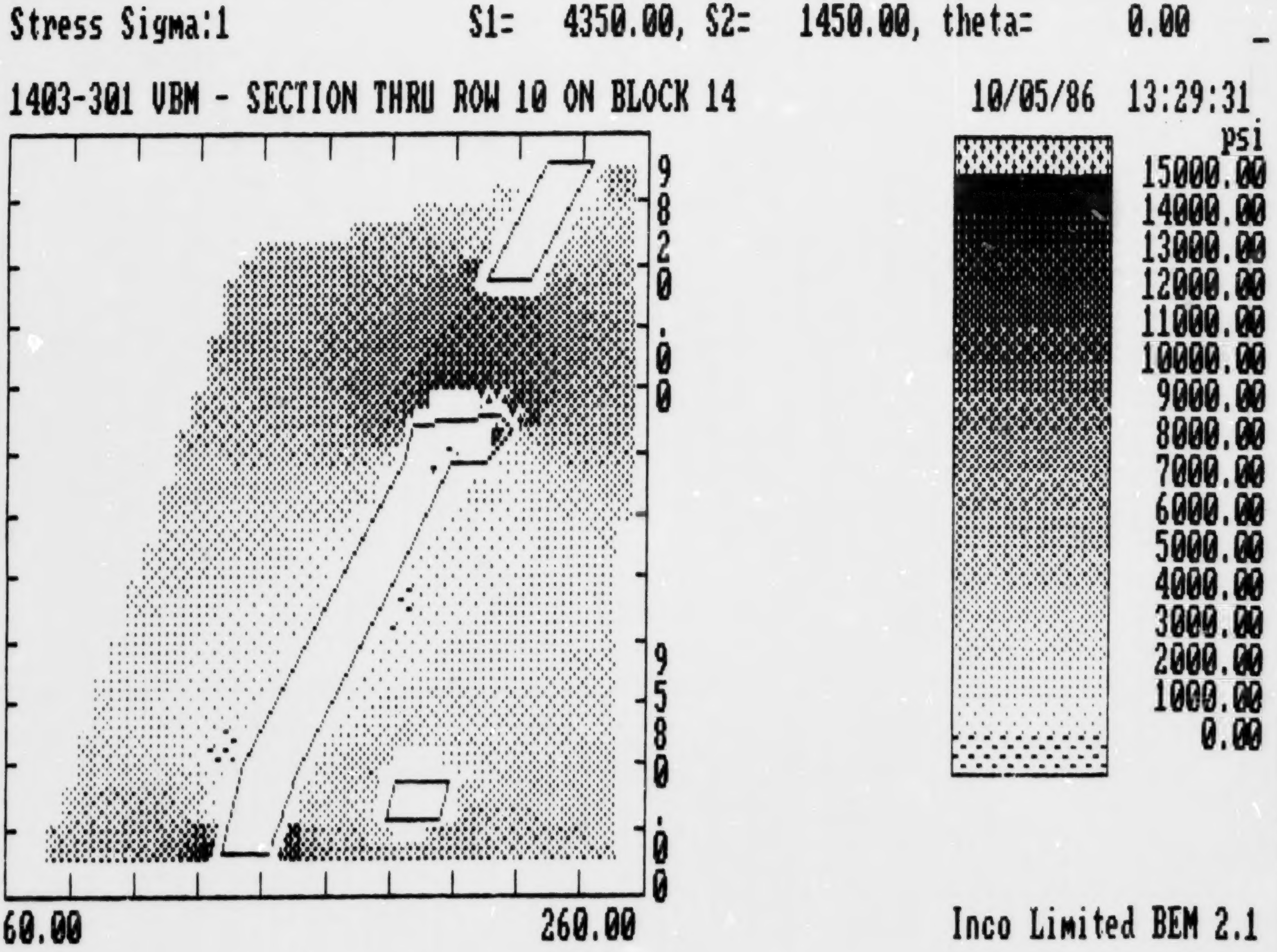
QUAD FINITE ELEMENT MESH

FIG. 3.62



BOUNDARY ELEMENT OUTPUT

FIG. 3.63

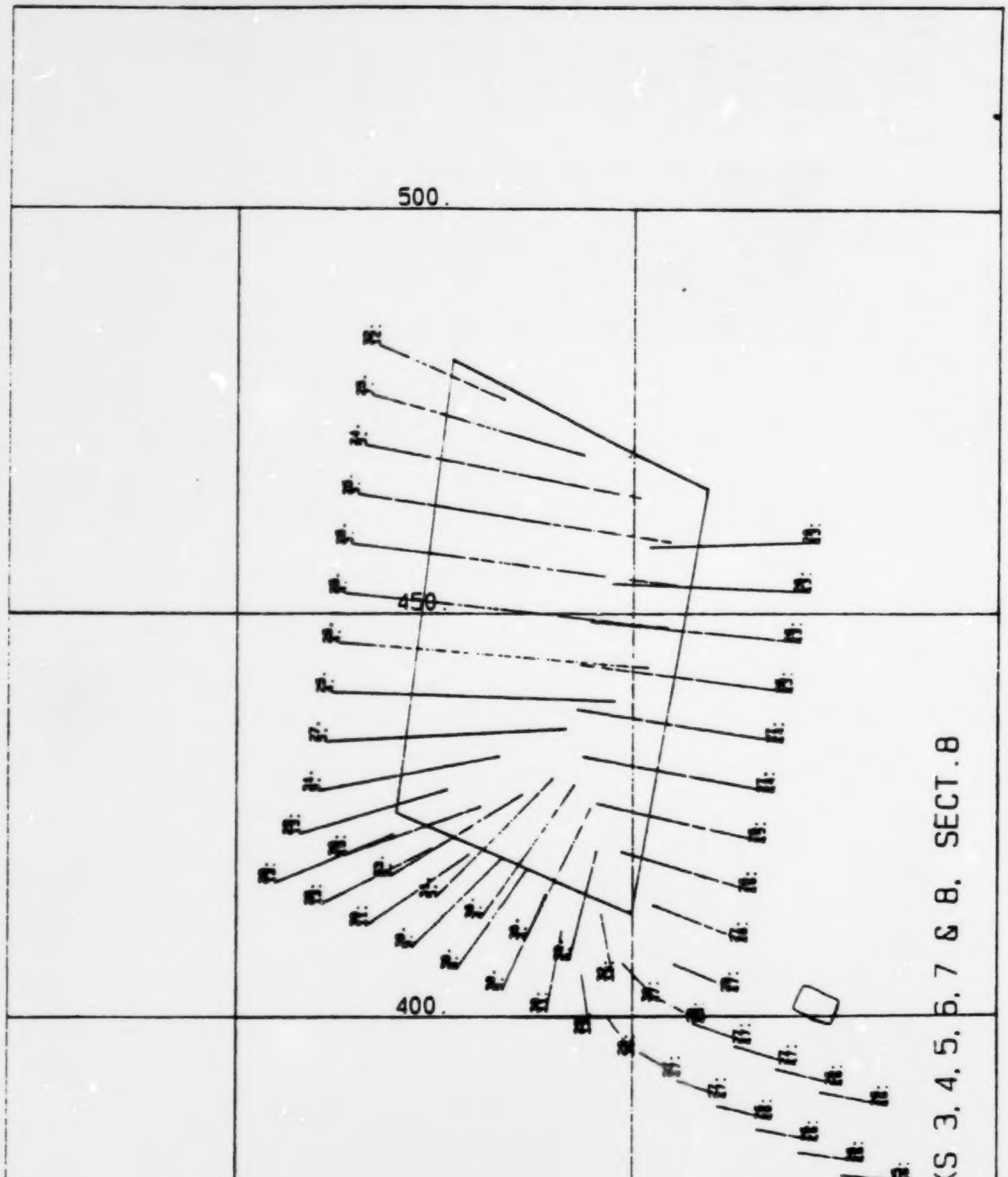

Stress Sigma:1 S1= 4350.00, S2= 1450.00, theta= 0.00

1403-301 VBM - SECTION THRU ROW 10 ON BLOCK 14 10/05/86 13:29:31

Inco Limited BEM 2.1

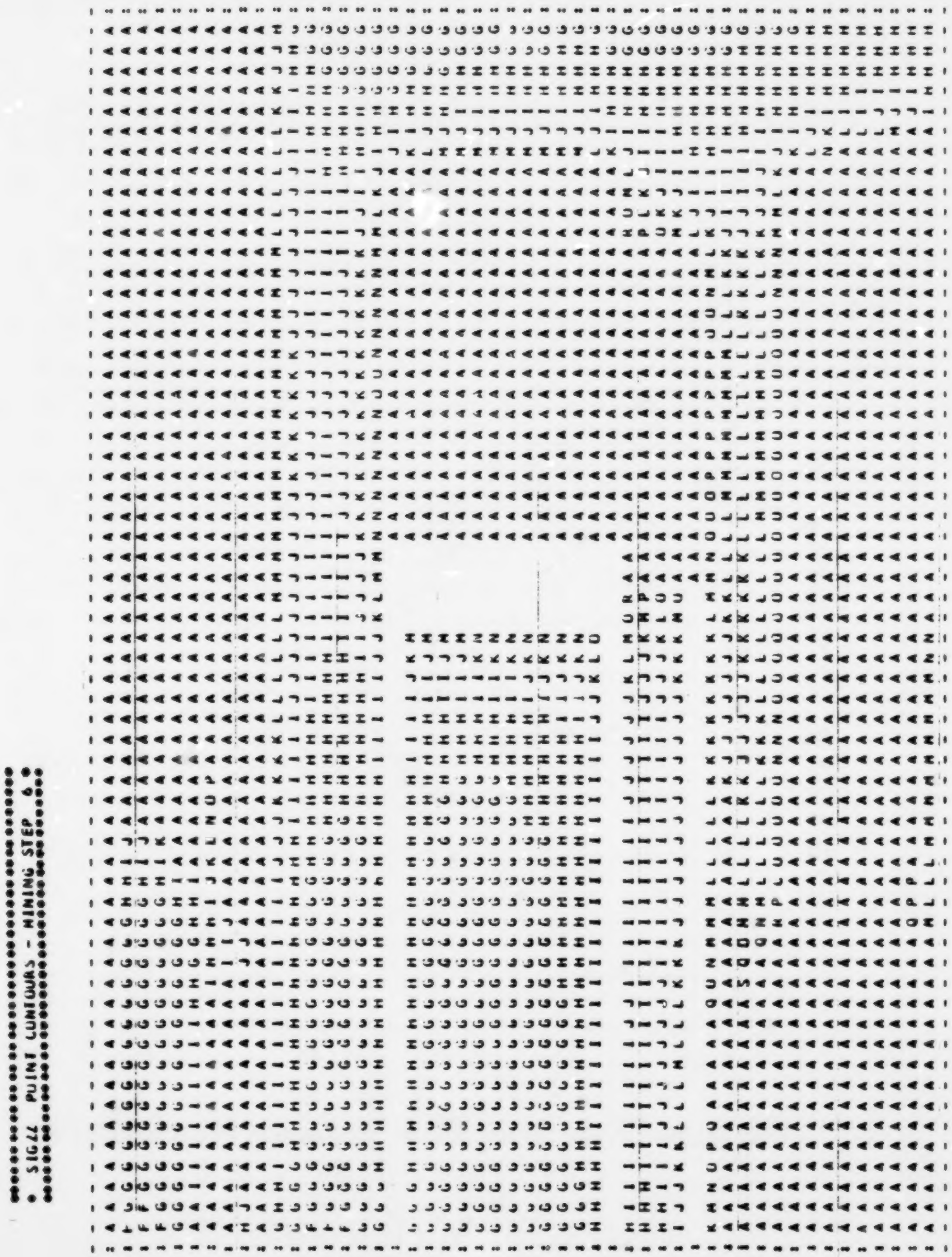
M – BEM OUTPUT

FIG. 3.64



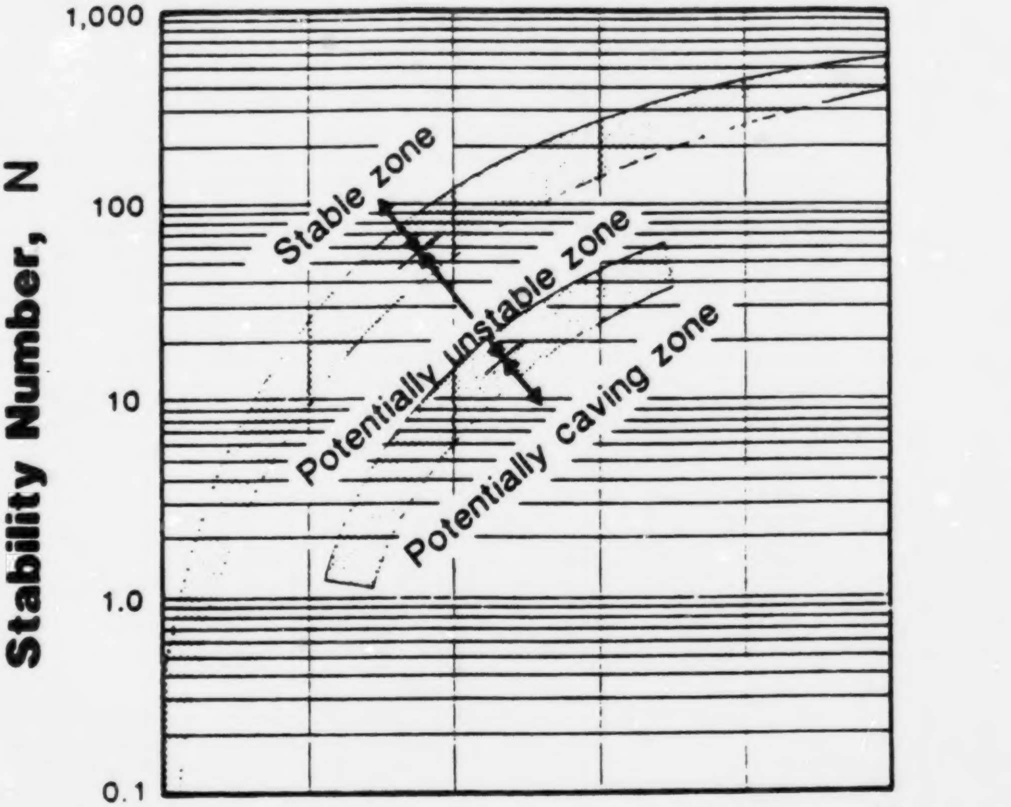
MINTAB OUTPUT
1803 — 331 STOPE

FIG. 3.65



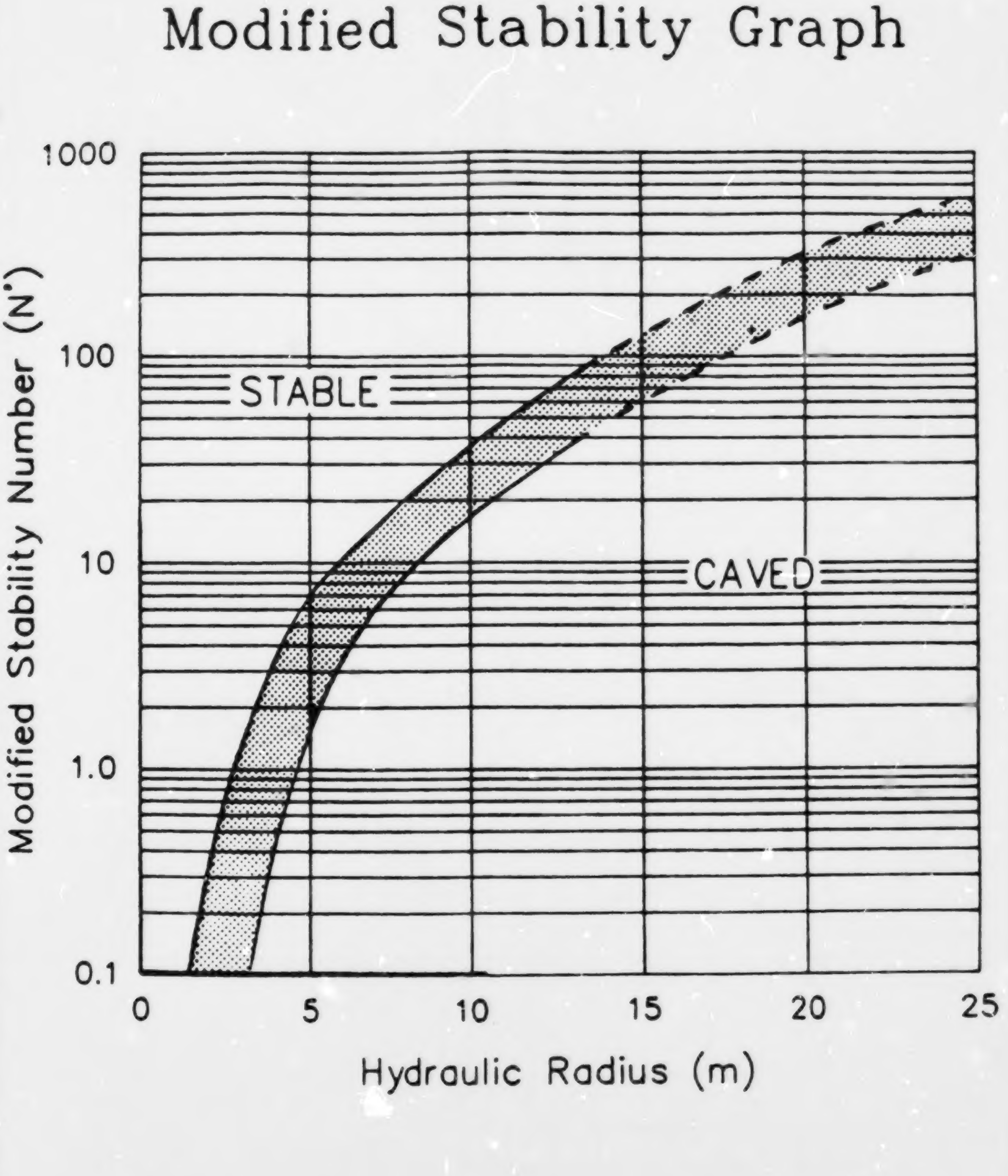
STABILITY GRAPH

FIG. 3.66



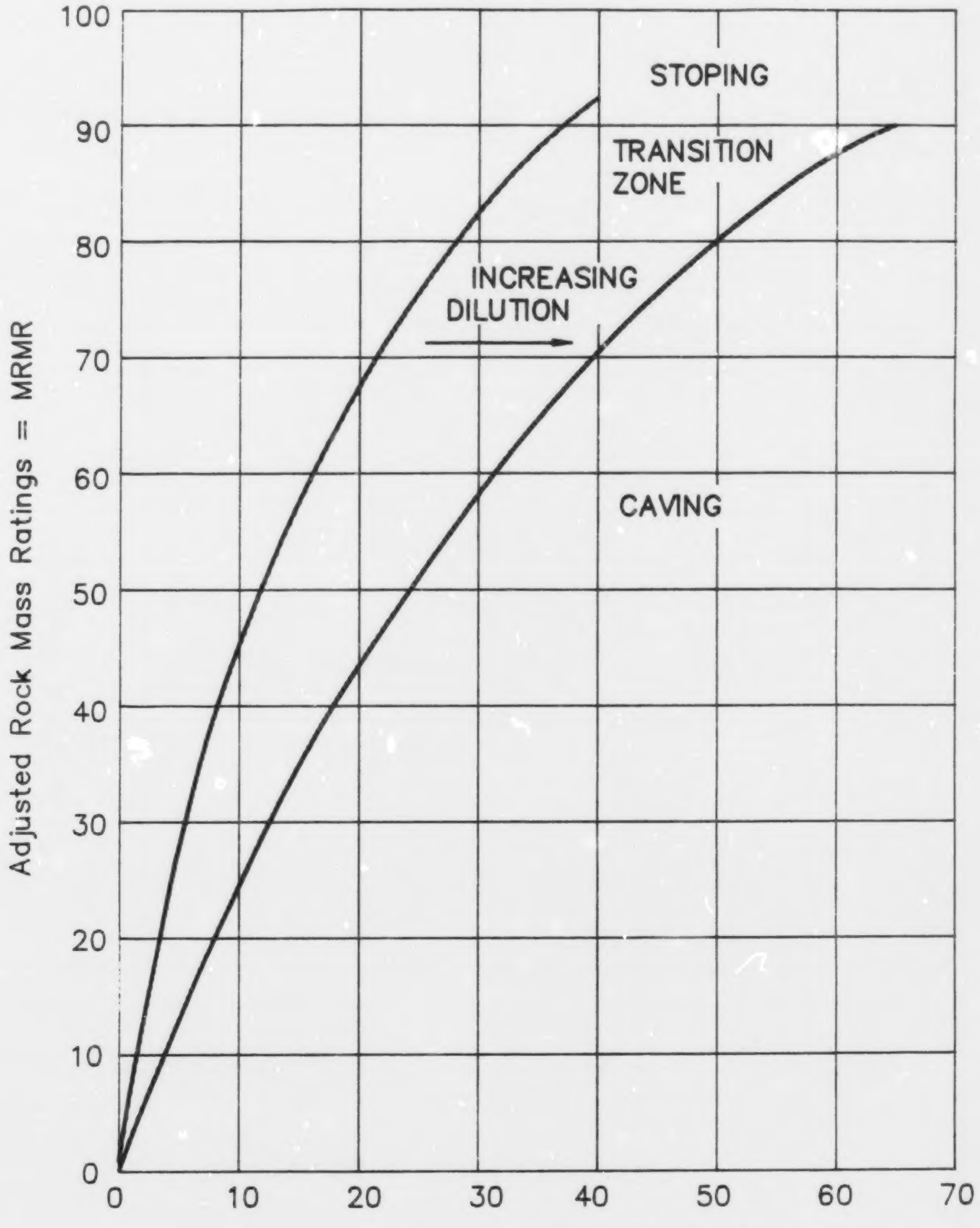
MODIFIED STABILITY GRAPH

FIG. 3.67



MRMR VS. HYDRAULIC RADIUS

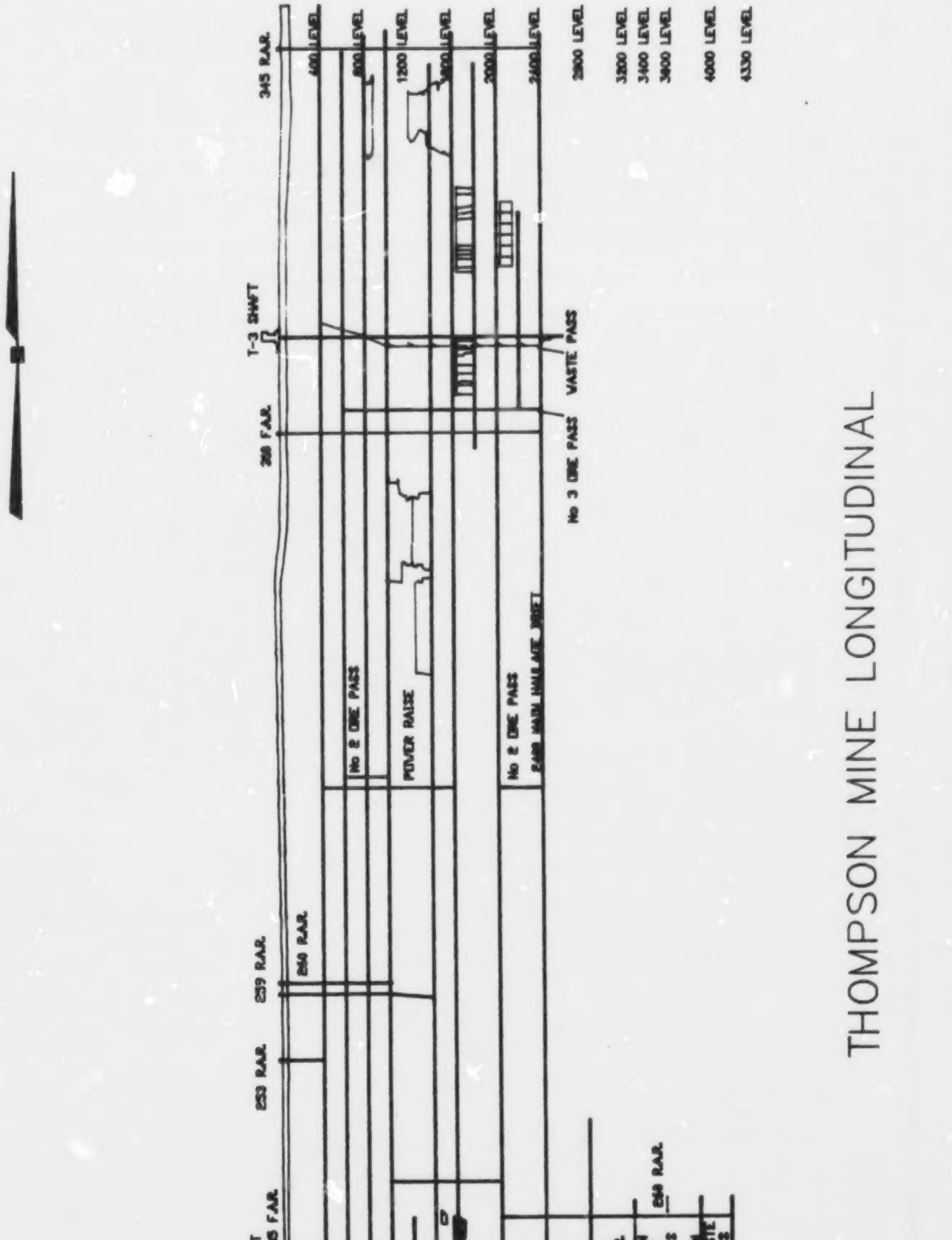
FIG. 3.68



(after Laubscher,1987)

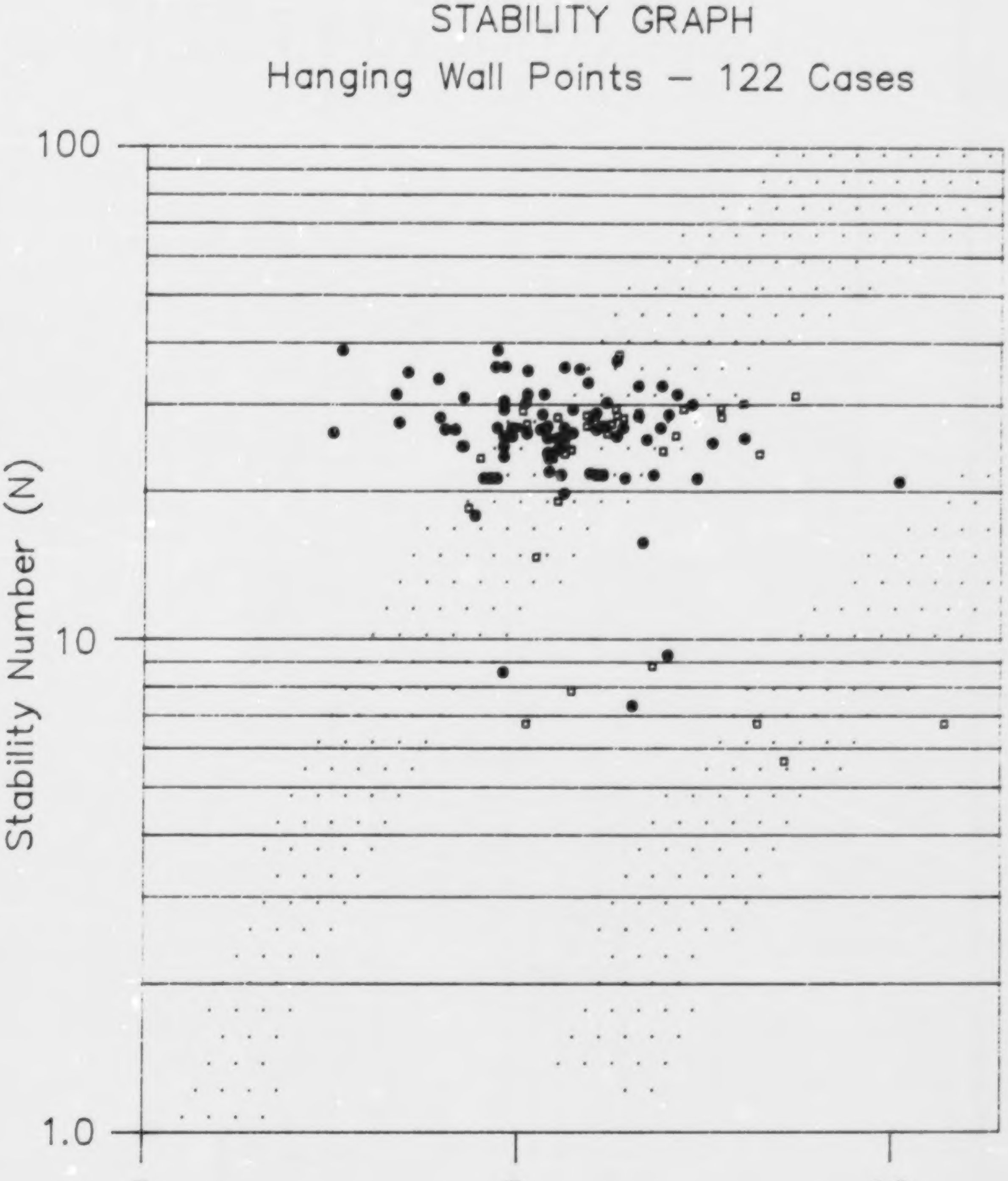
THOMPSON MINE LONGITUDINAL SHOWING TRIAL VBM STOPES

FIG. 3.69


345 R.A.R.
400 LEVEL
800 LEVEL
1200 LEVEL
1600 LEVEL
2000 LEVEL
2400 LEVEL
2800 LEVEL
3200 LEVEL
3400 LEVEL
3600 LEVEL
4000 LEVEL
4330 LEVEL
T-3 SHAFT
WASTE PASS
268 F.A.R.
No 3 ORE PASS
No 2 ORE PASS
POWER RAISE
No 2 ORE PASS
2400 MAIN HAULAGE DRIFT
260 R.A.R.
259 R.A.R.
253 R.A.R.
T-1 SHAFT
234-235 F.A.R.
235 F.A.R.
MAIN ORE PASS
268 R.A.R.
MAIN WASTE PASS
THOMPSON MINE LONGITUDINAL

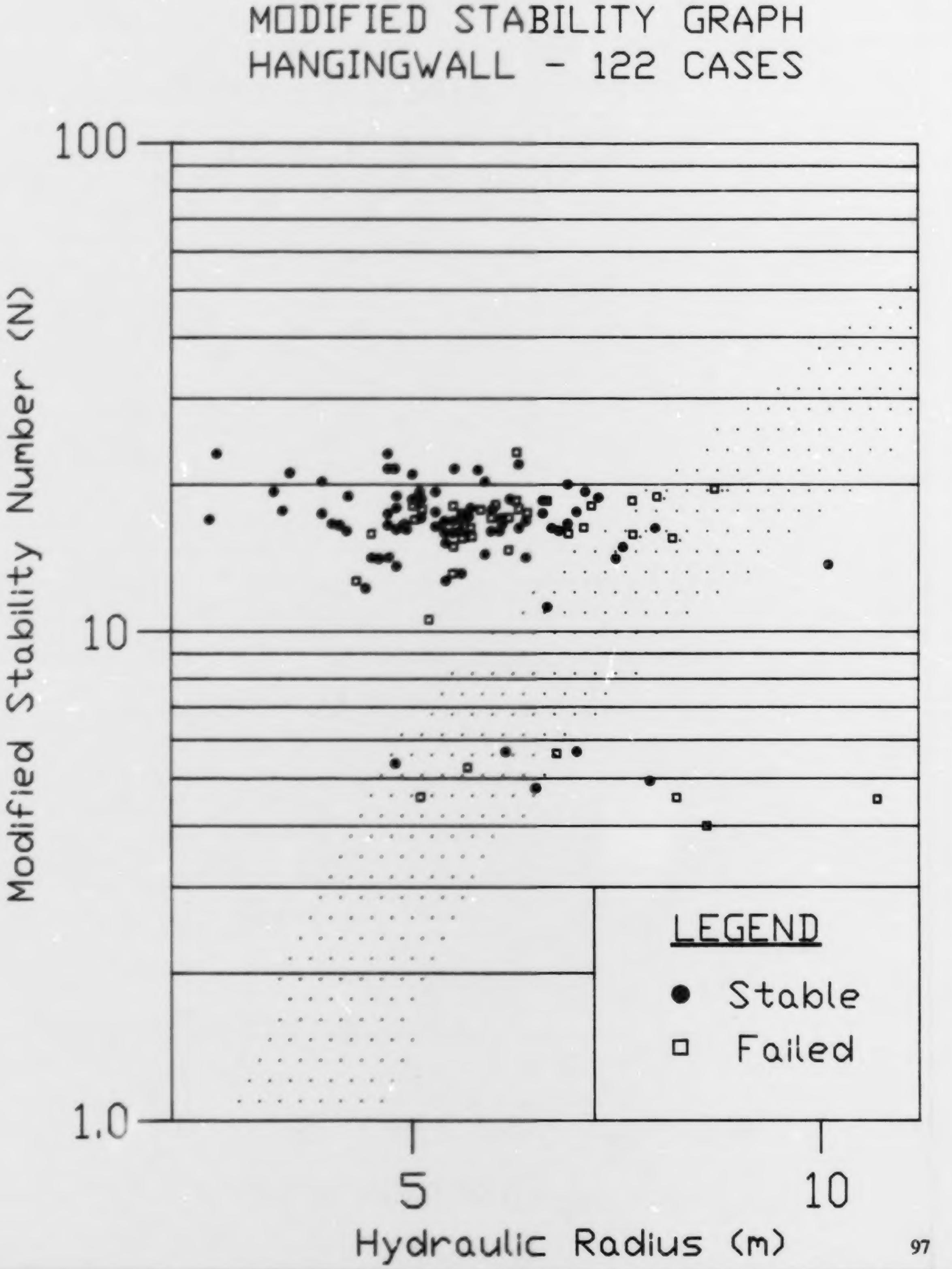
STABILITY GRAPH HANGINGWALL EXPERIENCE

FIG. 3.70



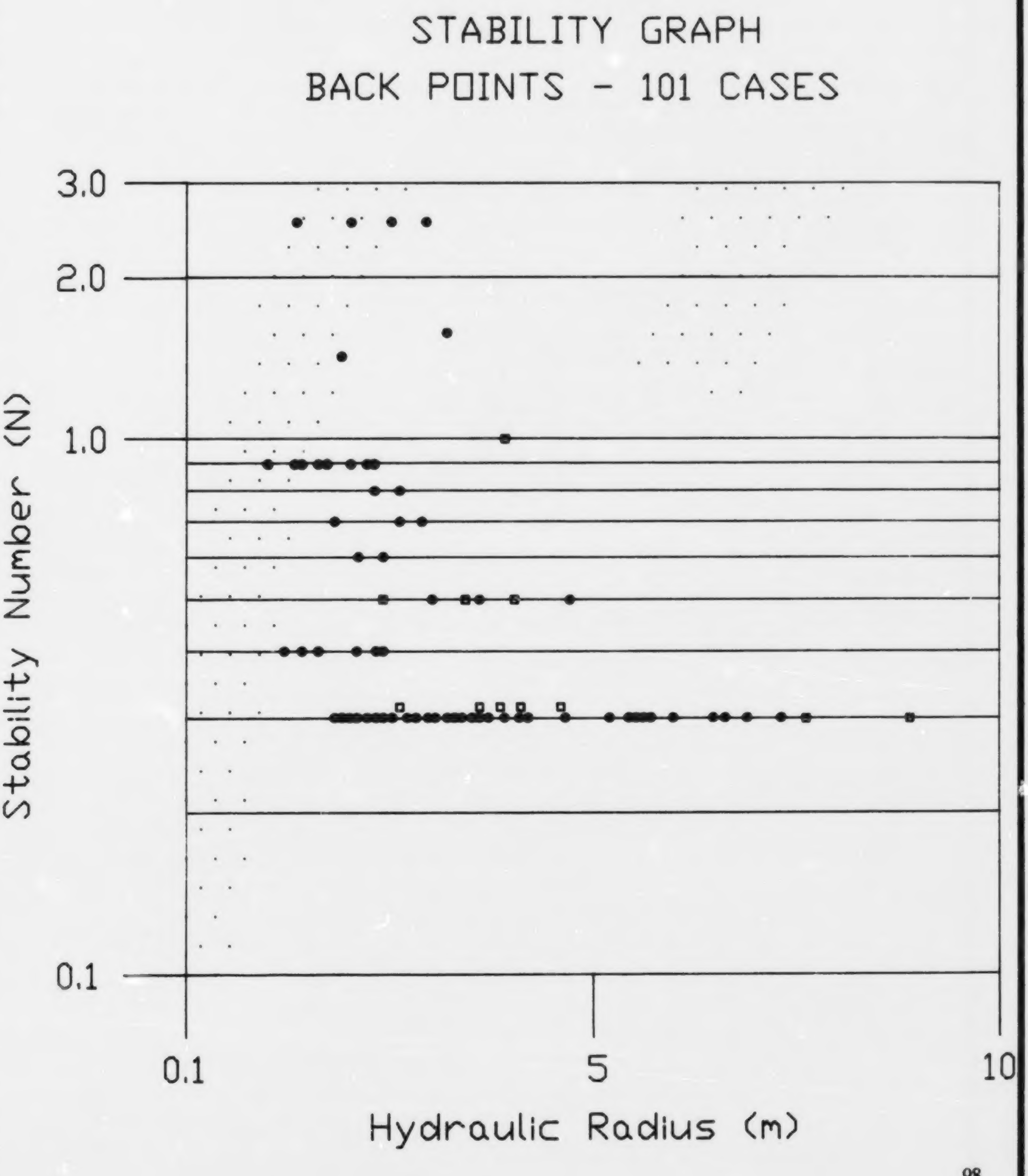
MODIFIED STABILITY GRAPH
HANGINGWALL EXPERIENCE

FIG. 3.71



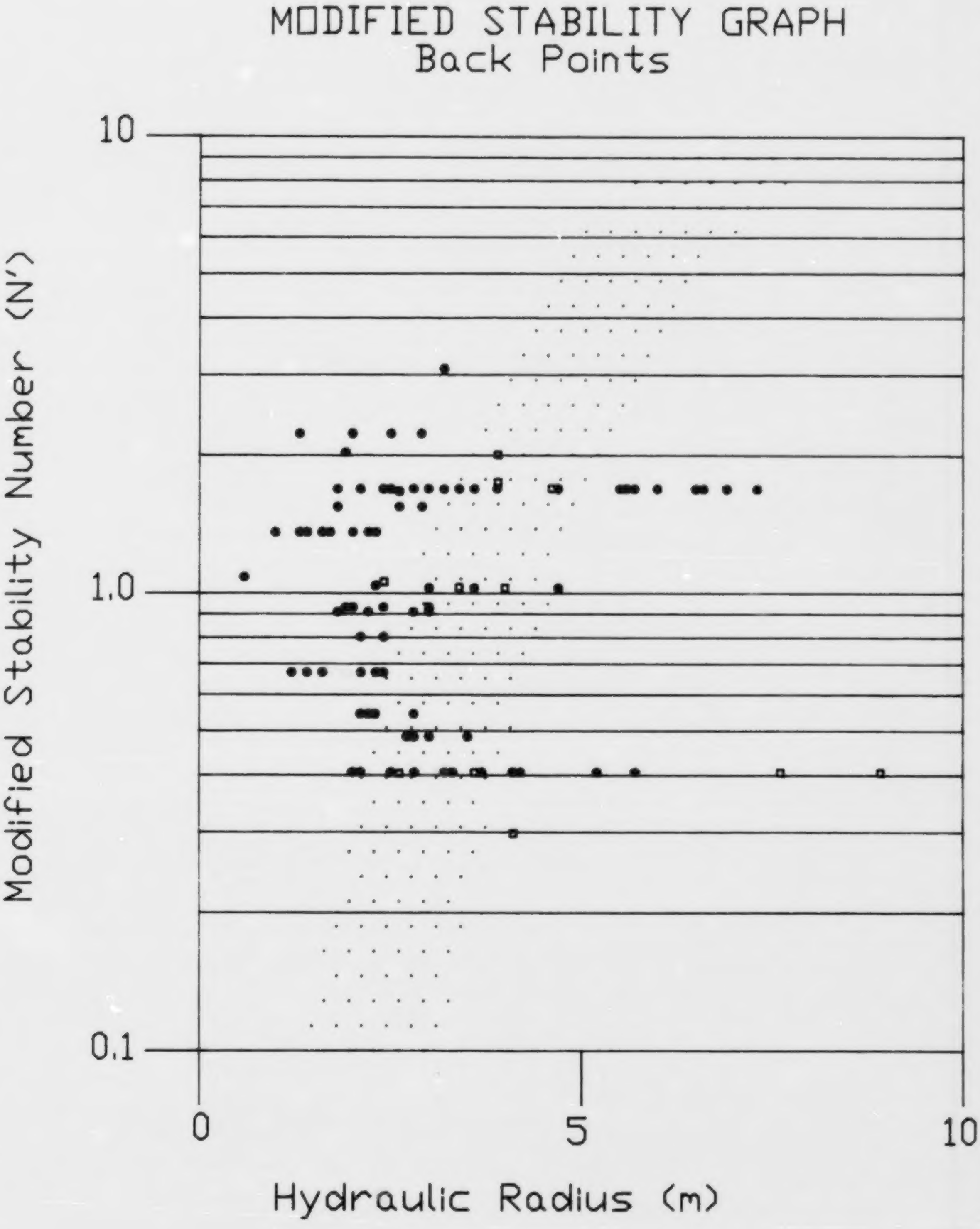
STABILITY GRAPH BACK EXPERIENCE

FIG. 3.72



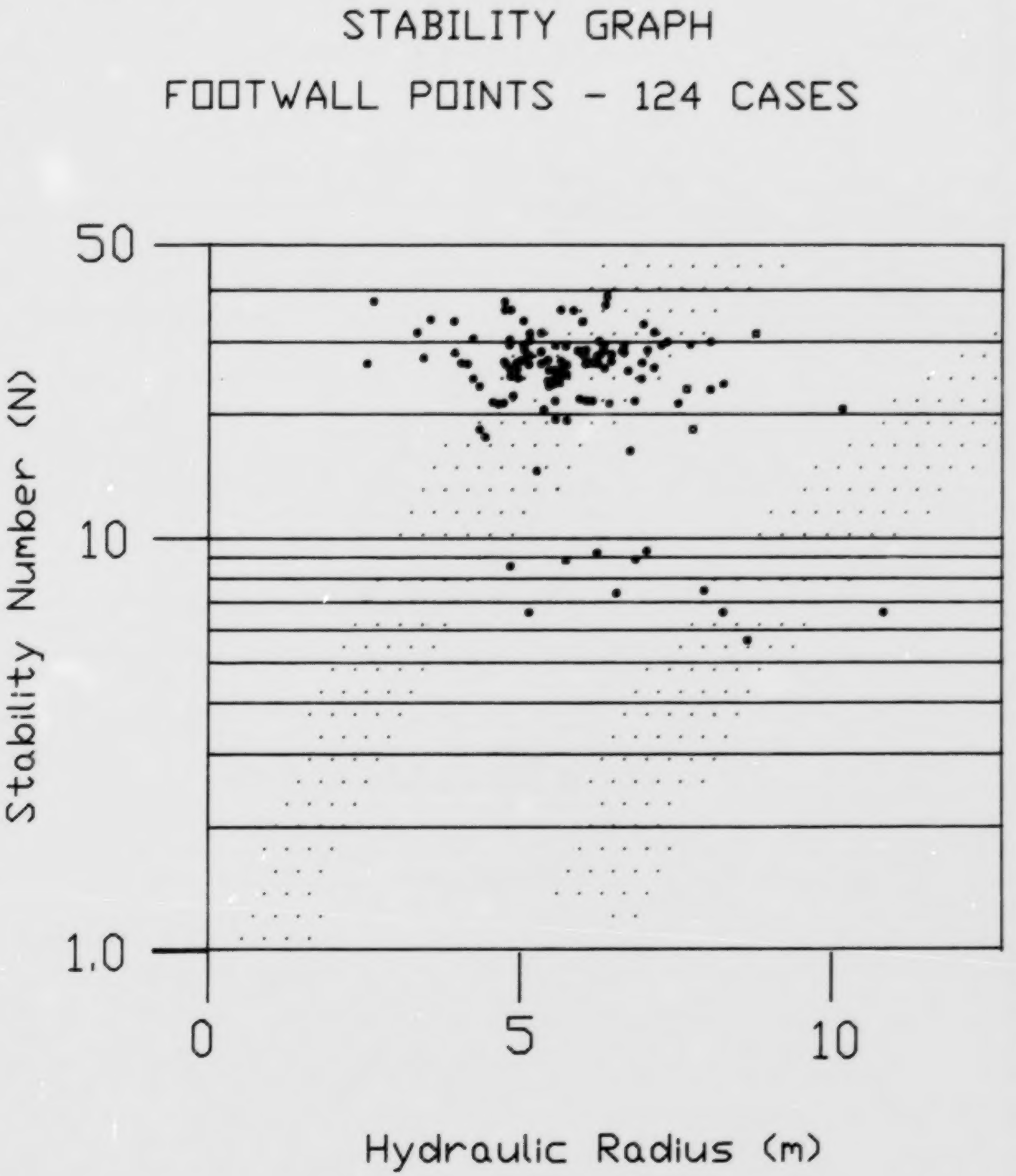
MODIFIED STABILITY GRAPH BACK EXPERIENCE

FIG. 3.73



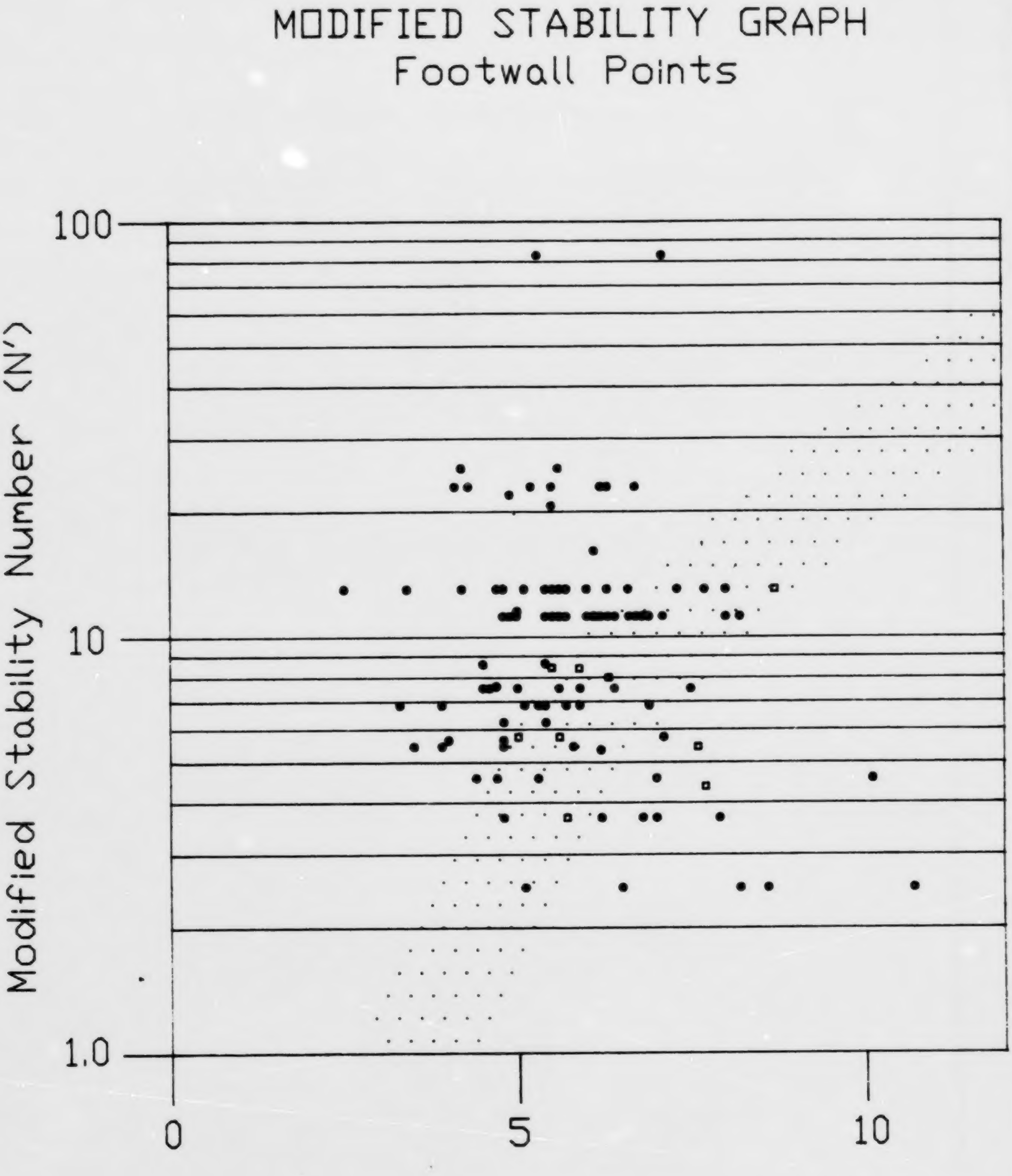
STABILITY GRAPH FOOTWALL EXPERIENCE

FIG. 3.74



MODIFIED STABILITY GRAPH FOOTWALL EXPERIENCE

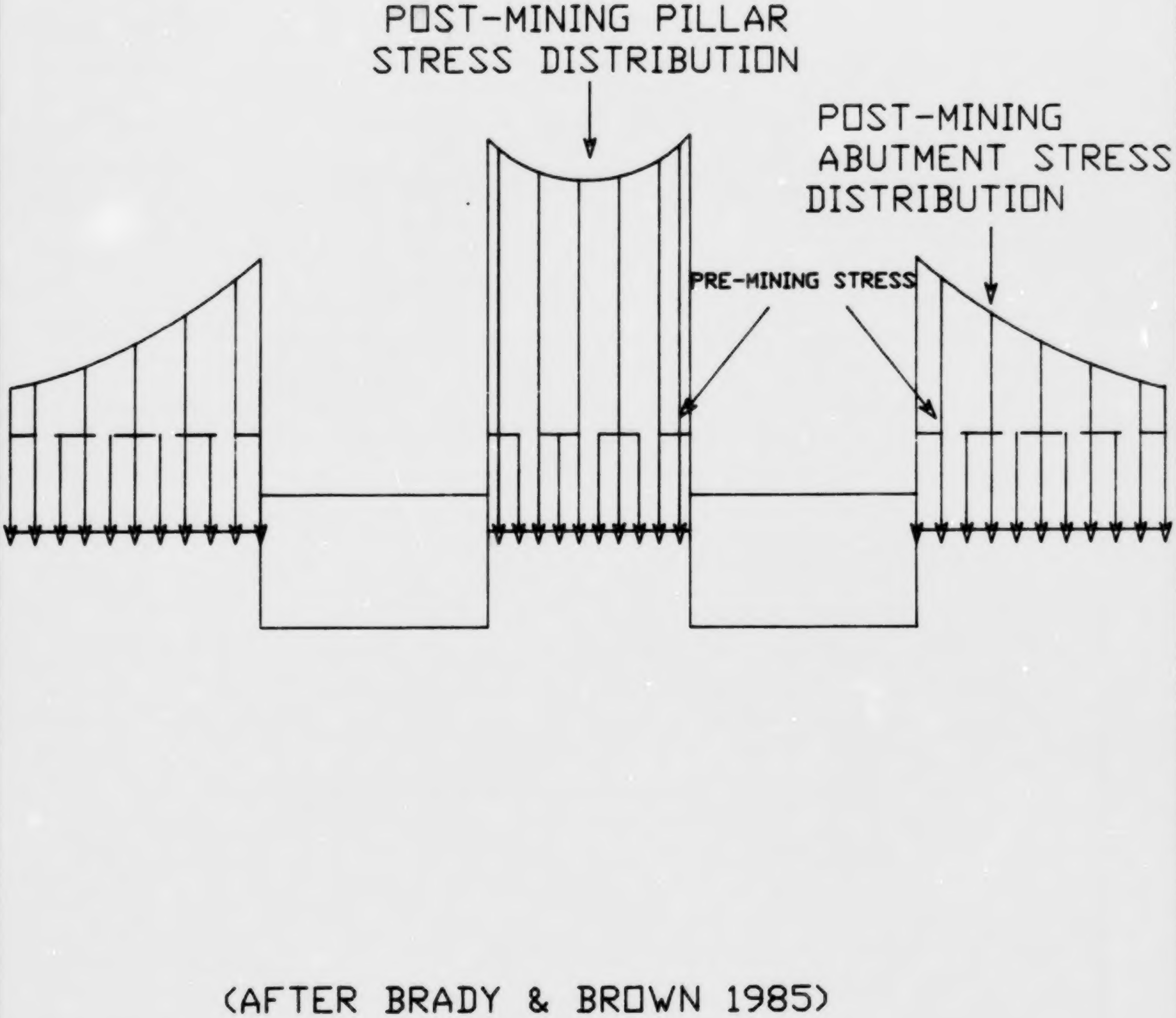
FIG. 3.75



Hydraulic Radius (m)

REDISTRIBUTION OF STRESS IN PILLAR

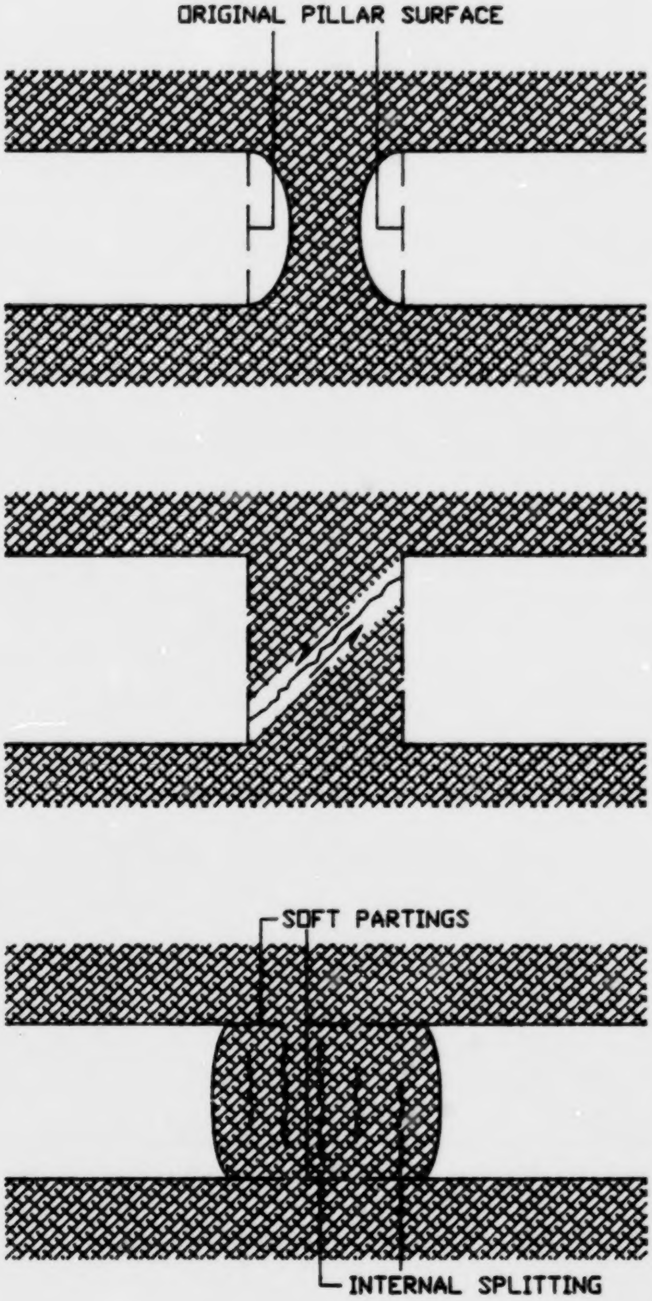
FIG. 3.76


POST-MINING PILLAR STRESS DISTRIBUTION
POST-MINING ABUTMENT STRESS DISTRIBUTION
PRE-MINING STRESS

(AFTER BRADY & BROWN 1985)

THREE MAIN MODES OF DEFORMATION BEHAVIOUR

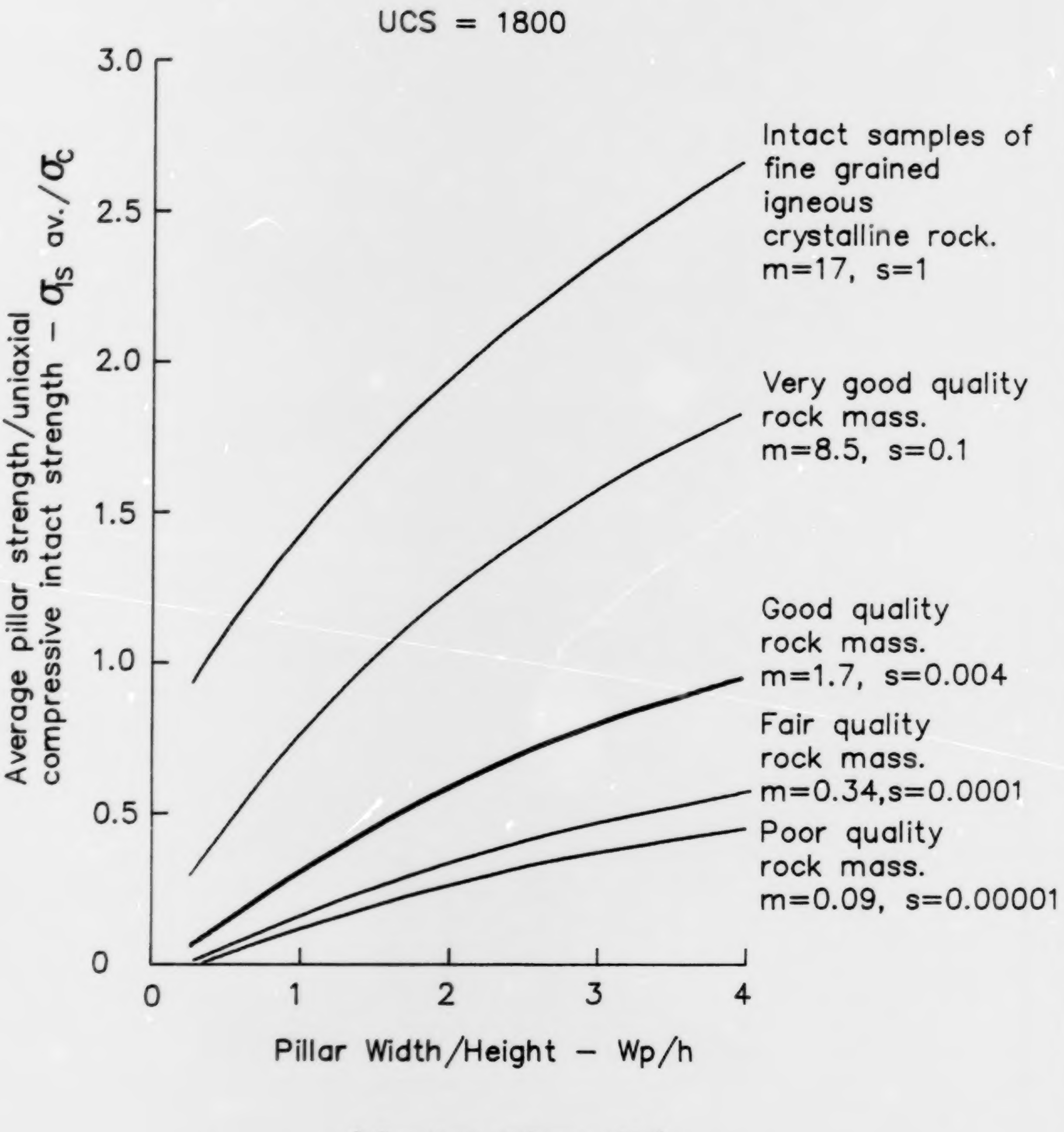
FIG. 3.77


ORIGINAL PILLAR SURFACE
SOFT PARTINGS
INTERNAL SPLITTING

(AFTER BRADY & BROWN 1985)

INFLUENCE OF PILLAR WIDTH TO HEIGHT RATIO ON AVERAGE STRENGTH

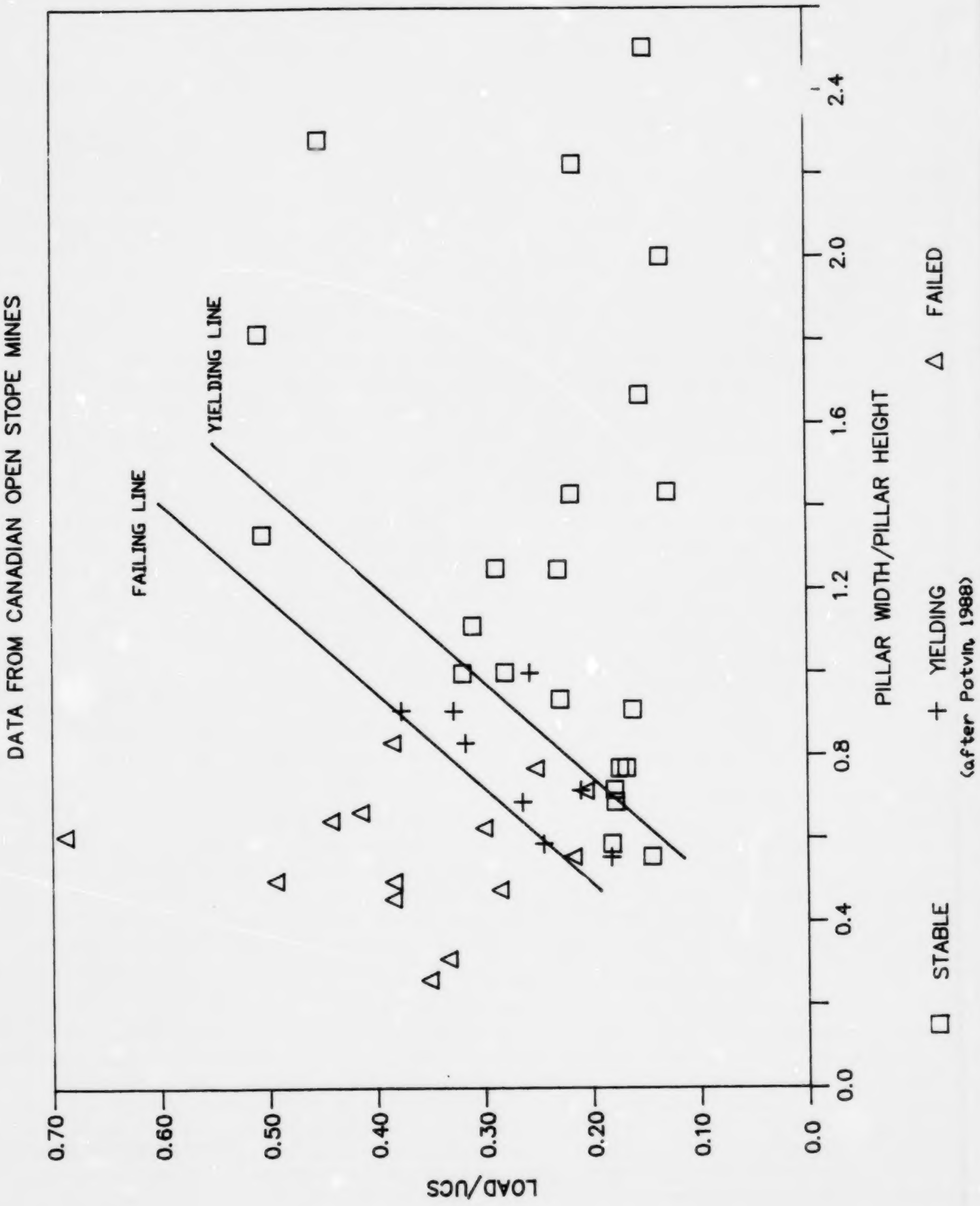
FIG. 3.78



(after Hoek & Brown,1980)

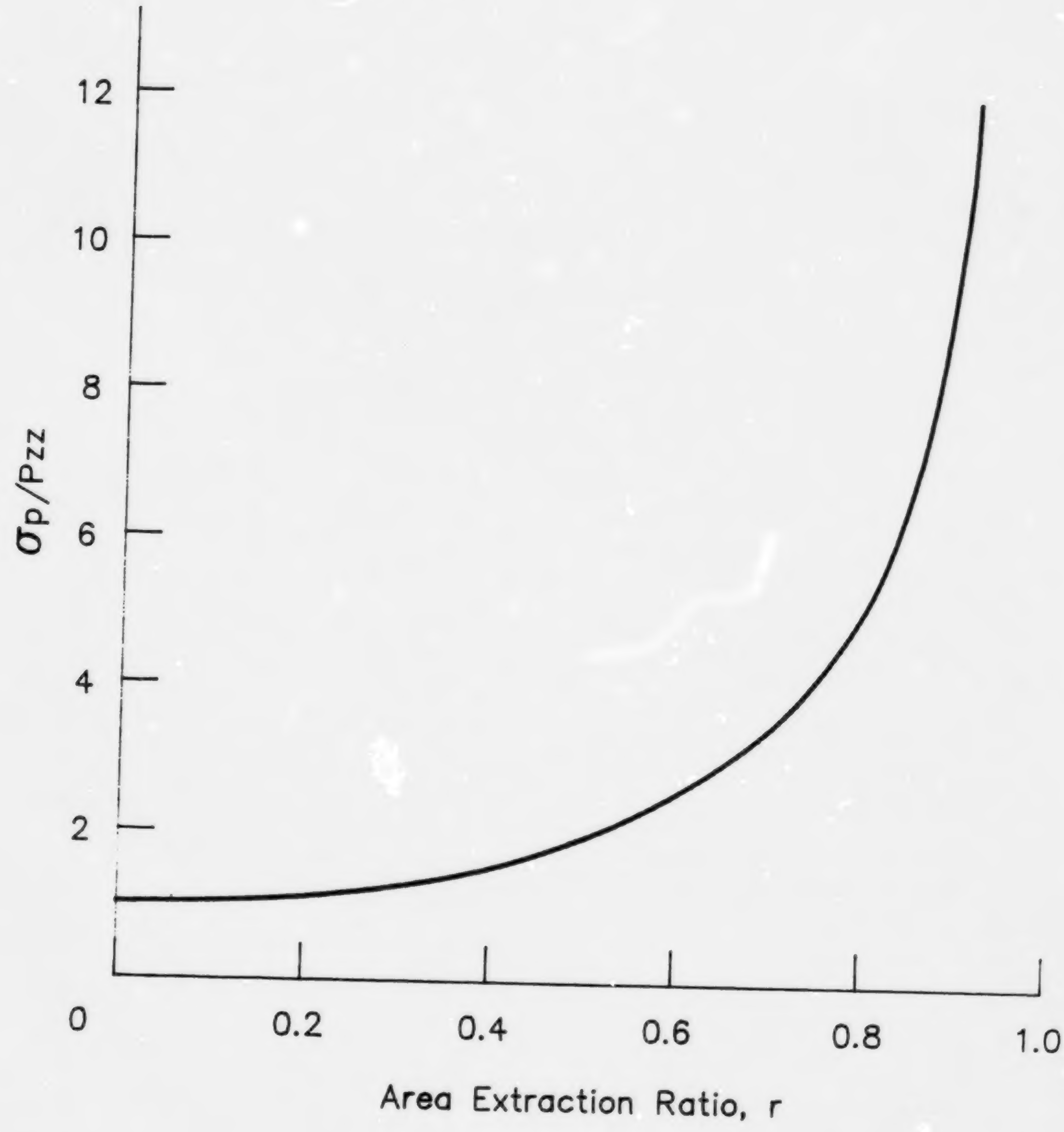
PILLAR STABILITY GRAPH

FIG. 3.79



LOAD VS. EXTRACTION RATIO

FIG. 3.80



(after Brady & Brown,1985)

MINTAB SEQUENCING ANALYSIS
SCENARIO No 1

FIG. 3.81

```
     CHCK
        * 2.1  MINE NAME     FORMAT(A80) RECORDS 2.1-2.3
     THOMPSON MINE - 1C ZONE
        * 2.2  JOB DESCRIPTION
       1C MINING SCENARIO NO. 1 - CLEAN SWEEP.
0000000000000000011111111111111111111111110000000000000000000000000000
0000000000000000111111111111111111111111110000000000000000000000000000
0000000000000000111111111111111111111111110000000000000000000000000000
0000000000000001111111111111111111111111110000000000000000000000000000
0000000000000001111111111111111111111111111111100000000000000000000000
0000000000000011111111111111111111111111110000000000000000000000000000
0000000000000111111111111111111111111111110000000000000000000000000000
0000000000000111111111111111111111111111111110000000000000000000000000
0000000000001111111111111111111111111111111111100000000000000000000000
0000000000001111111111111111111111111111111111100000000000000000000000
0000022222000000000WVVVU0000000000000000000000000000000000000000000000
000002222200000000WWVVVUU000000000000000000000000000000000000000000000
00000222220000000WWWVVVUUU00000000000000000000000000000000000000000000
0000022222000000WWWWVVVUUUSSSRRRP0000000000000000000000000000000000000
0000022222222222WWWWVVVUUUSSSRRRPP000000000000000000000000000000000000
0000022222222222WWWWVVVUUUSSSRRRPP000000000000000000000000000000000000
0000022222222222WWWWVVVUUUSSSRRRPPP00000000000000000000000000000000000
0000000000022222222UUUUTTTRRRQQQOOO00000000000000000000000000000000000
0000000000022222222UUUUTTTRRRQQQOOON0000000000000000000000000000000000
0000000000022222222UUUUTTTRRRQQQOOONNNLLLJJJHHHFF000000000000000000000
0000000000022222222UUUUTTTRRRQQQOOONNNLLLJJJHHHFFFC0000000000000000000
0000000000022222222UUUUTTTRRRQQQOOONNNLLLJJJHHHFFFCC000000000000000000
0000000000000000000SSSSQQQPPPOOONNNMMMKKKIIIGGGEEECCCAAA88000000000000
00000000000000000000SSSQQQPPPOOONNNMMMKKKIIIGGGEEECCCAAA88800000000000
00000000000000000000SSSQQQPPPOOONNNMMMKKKIIIGGGEEECCCAAA88800000000000
000000000000000000000SSQQQPPPOOONNNMMMKKKIIIGGGEEECCCAAA88360000000000
000000000000000000000SSQQQPPPOOONNNMMMKKKIIIGGGEEECCCAAA88860000000000
000000000000000000000000000PPMMMLLLKKKJJJHHHFFFDDDBBB99977766000000000
0000000000000000000000000000PMMMLLLKKKJJJHHHFFFDDDBBB99977766000000000
00000000000000000000000000000MMMLLLKKKJJJHHHFFFDDDBBB99977766000000000
000000000000000000000000000000MMLLLKKKJJJHHHFFFDDDBBB99977766600000000
0000000000000000000000000000000MLLLKKKJJJHHHFFFDDDBBB99977766600000000
0000000000000000000000000000000000000KIIIGGGEEECCCAAA88866655530000000
00000000000000000000000000000000000000IIIGGGEEECCCAAA88866655530000000
00000000000000000000000000000000000000IIIGGGEEECCCAAA88866655533000000
000000000000000000000000000000000000000IIGGGEEECCCAAA88866655533000000
000000000000000000000000000000000000000IIGGGEEECCCAAA88866655533000000
0000000000000000000000000000000000000000IFFFDDDBBB99977755544433300000
0000000000000000000000000000000000000000IFFFDDDBBB99977755544433300000
00000000000000000000000000000000000000000FFFDDDBBB99977755544433300000
000000000000000000000000000000000000000000FFDDDBBB99977755544433300000
00000000000000000000000000000000000000000000DDDBBB99977755544433300000
000000000000000000000000000000000000000000000000BB99977755544433300000
0000000000000000000000000000000000000000000000000000000000044433300000
0000000000000000000000000000000000000000000000000000000000044433300000
0000000000000000000000000000000000000000000000000000000000004433000000
0000000000000000000000000000000000000000000000000000000000000430000000
0000000000000000000000000000000000000000000000000000000000000000000000
0000000000000000000000000000000000000000000000000000000000000000000000
0000000000000000000000000000000000000000000000000000000000000000000000
```

MINTAB SEQUENCING ANALYSIS
SCENARIO No 2

FIG. 3.82

```
STRT
   * 2.1  MINE NAME      FORMAT(A80) RECORDS 2.1-2.3
THOMPSON MINE - 1C ZONE
   * 2.2  JOB DESCRIPTION
  1C MINING SCENARIO NO. 2 - PRIMARY SECONDARY METHOD W\ QUICK PILLAR RE
 0000000000000000011111111111111111111111110000000000000000000000000000
 0000000000000000111111111111111111111111110000000000000000000000000000
 0000000000000000111111111111111111111111110000000000000000000000000000
 0000000000000001111111111111111111111111110000000000000000000000000000
 0000000000000001111111111111111111111111111111100000000000000000000000
 0000000000000011111111111111111111111111111100000000000000000000000000
 0000000000000111111111111111111111111111111100000000000000000000000000
 0000000000000111111111111111111111111111111111000000000000000000000000
 0000000000001111111111111111111111111111111111100000000000000000000000
 0000000000001111111111111111111111111111111111100000000000000000000000
 0000022222000000000SFPPV0000000000000000000000000000000000000000000000
 000002222200000000SSFPPVV000000000000000000000000000000000000000000000
 00000222220000000SSSFPPVVV00000000000000000000000000000000000000000000
 0000022222000000SSSSFPPVVVMMMTTTR0000000000000000000000000000000000000
 0000022222222222SSSSFPPVVVMMMTTTRR000000000000000000000000000000000000
 0000022222222222SSSSFPPVVVMMMTTTRR000000000000000000000000000000000000
 0000022222222222SSSSFPPVVVMMMTTTRRR00000000000000000000000000000000000
 0000000000022222222000UUULLLSSSJJJ00000000000000000000000000000000000
 00000000000222222220000UUULLLSSSJJJQ0000000000000000000000000000000000
 00000000000222222220000UUULLLSSSJJJQQQGGGNNNFFFKK000000000000000000000
 00000000000222222220000UUULLLSSSJJJQQQGGGNNNFFFKKKA0000000000000000000
 00000000000222222220000UUULLLSSSJJJQQQGGGNNNFFFKKKAA000000000000000000
 00000000000000000000NNNNTTTKKRRRIIIPPEEEMMMDDDJJJAAAGGG770000000000000
 000000000000000000000NNNTTTKKRRRIIIPPEEEMMMDDDJJJAAAGGG777000000000000
 000000000000000000000NNNTTTKKRRRIIIPPEEEMMMDDDJJJAAAGGG777000000000000
 0000000000000000000000NNTTTKKRRRIIIPPEEEMMMDDDJJJAAAGGG777900000000000
 0000000000000000000000NNTTTKKRRRIIIPPEEEMMMDDDJJJAAAGGG777900000000000
 0000000000000000000000000000KKQQQHHHOOOBBBLLLCCCHHH999FFF6669000000000
 00000000000000000000000000000KQQQHHHOOOBBBLLLCCCHHH999FFF6669000000000
 000000000000000000000000000000QQQHHHOOOBBBLLLCCCHHH999FFF6669000000000
 0000000000000000000000000000000QQHHHOOOBBBLLLCCCHHH999FFF6669900000000
 00000000000000000000000000000000QHHHOOOBBBLLLCCCHHH999FFF6669900000000
 00000000000000000000000000000000000000AAAIII888EEE666DDD55588840000000
 00000000000000000000000000000000000000AAAIII888EEE666DDD55588840000000
 00000000000000000000000000000000000000AAAIII888EEE666DDD555888440000000
 000000000000000000000000000000000000000AAIII888EEE666DDD55588844000000
 000000000000000000000000000000000000000AAIII888EEE666DDD55588844000000
 0000000000000000000000000000000000000000AGGG777CCC555BBB44477733300000
 0000000000000000000000000000000000000000AGGG777CCC555BBB44477733300000
 00000000000000000000000000000000000000000GGG777CCC555BBB44477733300000
 000000000000000000000000000000000000000000GG777CCC555BBB44477733300000
 00000000000000000000000000000000000000000000777CCC555BBB44477733300000
 000000000000000000000000000000000000000000000000CC555BBB44477733300000
 0000000000000000000000000000000000000000000000000000000000077733300000
 0000000000000000000000000000000000000000000000000000000000077733300000
 0000000000000000000000000000000000000000000000000000000000007733000000
 0000000000000000000000000000000000000000000000000000000000000730000000
 0000000000000000000000000000000000000000000000000000000000000000000000
 0000000000000000000000000000000000000000000000000000000000000000000000
 0000000000000000000000000000000000000000000000000000000000000000000000
```

MINTAB SEQUENCING ANALYSIS
SCENARIO No 3

FIG. 3.83

```
STRT
   * 2.1   MINE NAME     FORMAT(A80) RECORDS 2.1-2.3
THOMPSON MINE - 1C ZONE
   * 2.2  JOB DESCRIPTION
  1C MINING SCENARIO NO. 3 - SIMPLE PRIMARY SECONDARY METHOD WITH DELA
0000000000000000011111111111111111111111110000000000000000000000000000
0000000000000000111111111111111111111111110000000000000000000000000000
0000000000000000111111111111111111111111110000000000000000000000000000
0000000000000001111111111111111111111111110000000000000000000000000000
0000000000000001111111111111111111111111111111100000000000000000000000
0000000000000011111111111111111111111111111100000000000000000000000000
0000000000000111111111111111111111111111111100000000000000000000000000
0000000000000111111111111111111111111111111111000000000000000000000000
0000000000001111111111111111111111111111111111100000000000000000000000
0000000000001111111111111111111111111111111111100000000000000000000000
0000022222000000000WHHHV0000000000000000000000000000000000000000000000
000002222200000000WWHHHVV000000000000000000000000000000000000000000000
00000222220000000WWWHHHVVV00000000000000000000000000000000000000000000
0000022222000000WWWWHHHVVVGGGUUUF0000000000000000000000000000000000000
0000022222222222WWWWHHHVVVGGGUUUFF000000000000000000000000000000000000
0000022222222222WWWWHHHVVVGGGUUUFF000000000000000000000000000000000000
0000022222222222WWWWHHHVVVGGGUUUFFF00000000000000000000000000000000000
0000000000022222222GGGGUUUFFFTTTEEE00000000000000000000000000000000000
0000000000022222222GGGGUUUFFFTTTEEER0000000000000000000000000000000000
0000000000022222222GGGGUUUFFFTTTEEERRRCCCQQQBBB00000000000000000000000
0000000000022222222GGGGUUUFFFTTTEEERRRCCCQQQBBB00080000000000000000000
0000000000022222222GGGGUUUFFFTTTEEERRRCCCQQQBBB00088000000000000000000
0000000000000000000EEEETTTDDDSSSDDDQQQBBBPPPAAANNN888LLL77000000000000
00000000000000000000EEETTTDDDSSSDDDQQQBBBPPPAAANNN888LLL77700000000000
00000000000000000000EEETTTDDDSSSDDDQQQBBBPPPAAANNN888LLL77700000000000
000000000000000000000EETTTDDDSSSDDDQQQBBBPPPAAANNN888LLL777J0000000000
000000000000000000000EETTTDDDSSSDDDQQQBBBPPPAAANNN888LLL777J0000000000
000000000000000000000000000DDRRRCCCPPPAAA000999MMM777KKK666JJ000000000
0000000000000000000000000000DRRRCCCPPPAAA000999MMM777KKK666JJ000000000
00000000000000000000000000000RRRCCCPPPAAA000999MMM777KKK666JJ000000000
000000000000000000000000000000RRCCCPPPAAA000999MMM777KKK666JJJ00000000
0000000000000000000000000000000RCCCPPPAAA000999MMM777KKK666JJJ00000000
0000000000000000000000000000000000000P999NNN888LLL666JJJ555III40000000
00000000000000000000000000000000000000999NNN888LLL666JJJ555III40000000
00000000000000000000000000000000000000999NNN888LLL666JJJ555III44000000
00000000000000000000000000000000000000099NNN888LLL666JJJ555III44000000
00000000000000000000000000000000000000099NNN888LLL666JJJ555III44000000
00000000000000000000000000000000000000009MMM777KKK555III444HHH33300000
00000000000000000000000000000000000000009MMM777KKK555III444HHH33300000
00000000000000000000000000000000000000000MMM777KKK555III444HHH33300000
000000000000000000000000000000000000000000MM777KKK555III444HHH33300000
00000000000000000000000000000000000000000000777KKK555III444HHH33300000
000000000000000000000000000000000000000000000000KK555III444HHH33300000
00000000000000000000000000000000000000000000000000000000000HHH33300000
00000000000000000000000000000000000000000000000000000000000HHH33300000
000000000000000000000000000000000000000000000000000000000000HH33000000
0000000000000000000000000000000000000000000000000000000000000H30000000
0000000000000000000000000000000000000000000000000000000000000000000000
0000000000000000000000000000000000000000000000000000000000000000000000
0000000000000000000000000000000000000000000000000000000000000000000000
```

MINTAB RESULTS—SCENARIO NO. 1
MINING STEP NO. 13

FIG. 3.84a

```
--------------------------------------------------------------------------------
:IIIIIIIIIIIJJJJKLAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAKKJJJJIIIIIIIIIIIIIIIIIIIIII:
:IIIIIIIIJJJJJKKMAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAMKKJJJJJIIIIIIIIIIIIIIIIIIII:
:IIIIIIJJJJJJKKLNAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAANLLKKJJJJIIIIIIIIIIIIIIIIIII:
:IIIIIJJJJJJKKLMAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAONMLKKJJJJJIIIIIIIIIIIIIIIII:
:IIIJJJJJJJKKLMOAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAKJJJJJJIIIIIIIIIIIIIIII:
:IJJJJJJJJKKLLNAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAOMLKKJJJJJJIIIIIIIIIIIIIII:
:IJJJJJJJKKKLMAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAOMLKKJJJJJJJJJJIIIIIIIIIII:
:JJJJJJKKKLLNAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAMLKJJJJJJJJJJJIIIIIIIIII:
:JJJJKKKKLLLMAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAALKJJJJJJJJJJJJJJJIIIIII:
:JJJKKLLMMMMMAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAALKKJJJJJJJJJJJJJJJJIIII:
:JJJKLAAAAAMMNNNCOCCOOOOOOOOOOOOOOOOOOOOOOOOOOOOONNNNMMLKKKJJJJJJJJJJJJJJJJJJJI:
:JJKKLAAAAANMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMMLLLLKKKKJJJJJJJJJJJJJJJJJJJJ:
:JJKKMAAAAAONNMMMMMMMMLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLKKKKKKKJJJJJJJJJJJJJJJJJJJJ:
:JJKKMAAAAAPOOONNMMLLLLLLLLLLLLLLLLLLLLLLLLLLLLLKKKKKKKKKKKJJJJJJJJJJJJJJJJJJJJ:
:JJKKMAAAAAAAAAAANMLLLLLLKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKJJJJJJJJJJJJJJJJJJJ:
:JJKKMAAAAAAAAAAAAOMMLLKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKJJJJJJJJJJJ:
:JJKKLAAAAAAAAAAAAPNMLLKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKJJJJJJJJJJJ:
:JJKKLMMNNOPAAAAAAAAMLKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKJJJJJJJ:
:JJKKKLLLMNOAAAAAAAAAMLLKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLLLLKKKKKKKJJJJJ:
:JJJKKKKLLMNAAAAAAAAAMLKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLMMLLLLLLLKKKKKKJJJJJ:
:JJJJKKKKLLNAAAAAAAAAMLKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLMANNNNMMMMLLKKKKKJJJJ:
:JJJJKKKKKLMAAAAAAAAAMLKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLMNAAPPPPOONMLLKKKKKKJJ:
:JJJJJKKKKKLMMNNNNMMLKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLMMOAAAAAAAAAPNMLLKKKKKJJ:
:JJJJJKKKKKKLLLLLLLLKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLMNPAAAAAAAAAAONMLLKKKKKJ:
:JJJJJKKKKKKKKLLLLKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLMMOPAAAAAAAAAAQOMMLLKKKKJ:
:JJJJJJKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLMMNPRAAAAAAAAAAAPNMMLKKKKK:
:JJJJJJJKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLMNPQSAAAAAAAAARONMLLKKKK:
:JJJJJJJKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLMNO    AAAAAAAAAAAAPNMMLLKKK:
:JJJJJJJKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLMNP    AAAAAAAAAAAAQONMLLKKK:
:JJJJJJKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLLMNQ    AAAAAAAAAAAASPNMLLLKK:
:JJJJJJJKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLMMOQ    AAAAAAAAAAAAAQONMLLKK:
:JJJJJJJKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLMNOR   AAAAAAAAAAAAASPNMLLLK:
:JJJJJKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLMMNORAAAAAAAAAAAAAAAAQOMMLLK:
:JJJJKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLMMNPRAAAAAAAAAAAAAAARPNMLLLK:
:JKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLMMNPRAAAAAAAAAAAAAAAAAQNMMLK:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLMNOPSAAAAAAAAAAAAAAAAAQONMLL:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLMMNPRTAAAAAAAAAAAAAAAARONMLL:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLMNO  AAAAAAAAAAAAAAAAAAPNMLL:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLMNO  AAAAAAAAAAAAAAAAAAQNMLL:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLMNP  AAAAAAAAAAAAAAAAAAQNMLL:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLMNO  AAAAAAAAAAAAAAAAAAQNMLL:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLMMO  AAAAAAAAAAAAAAAAAAPNMLL:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLLMNOPQRAAAAAAAAAAAAAAAAAPNMLL:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLLMMNNOPPQRRRRSSSTAAAAAAPNMLL:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLLLLMMMNNOOOPPPPPQSAAAAAAONMLL:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLLLLLMMMMNNNNNNOOOPQAAAAPNMLLL:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLLLLLMMMMMMMNNNNNOOPAAPNMMLLL:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLLLLLLLMMMMMMMMMMNNNOONMMLLLL:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLLLLLLLLLMMMMMMMMMMMMMMMLLLLL:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLLLLLLLLLLLLLLMMMMMMMMMLLLLLL:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLLLLLLLLLLLLLLLLLLLLLLLLLLLK:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLLLLLLLLLLLLLLLLLLLLLLLLLLLK:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLLLLLLLLLLLLLLLLLLLLLLLLLLLK:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLLLLLLLLLLLLLLLLLLLLLLLLLLLK:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLLLLLLLLLLLLLLLLLLLLLLLLLLLK:
--------------------------------------------------------------------------------
```

MINTAB RESULTS—SCENARIO NO. 1
MINING STEP NO. 13

FIG. 3.84b

```
*******************************************
*  SIG1  PRINT CONTOURS - MINING STEP 13 *
*******************************************


***** SYMBOL *****     ***** VALUE RANGE *****

        A              0.0          - 0.3500E 01
        B              0.3500E 01 - 0.7000E 01
        C              0.7000E 01 - 0.1050E 02
        D              0.1050E 02 - 0.1400E 02
        E              0.1400E 02 - 0.1750E 02
        F              0.1750E 02 - 0.2100E 02
        G              0.2100E 02 - 0.2450E 02
        H              0.2450E 02 - 0.2800E 02
        I              0.2800E 02 - 0.3150E 02
        J              0.3150E 02 - 0.3500E 02
        K              0.3500E 02 - 0.3850E 02
        L              0.3850E 02 - 0.4200E 02
        M              0.4200E 02 - 0.4550E 02
        N              0.4550E 02 - 0.4900E 02
        O              0.4900E 02 - 0.5250E 02
        P              0.5250E 02 - 0.5600E 02
        Q              0.5600E 02 - 0.5950E 02
        R              0.5950E 02 - 0.6300E 02
        S              0.6300E 02 - 0.6650E 02
        T              0.6650E 02 - 0.7000E 02

MINIMUM VALUE    =    0.234990E-02 MPA
MAXIMUM VALUE    =    0.697340E 02 MPA
CONTOUR INTERVAL =    0.350000E 01 MPA
```

MINTAB RESULTS—SCENARIO NO. 2
MINING STEP NO. 7

FIG. 3.85a

```
------------------------------------------------------------------------
:IIIIIIIIIIJJJJKLAAAAAAAAAAAAAAAAAAAAAAAAAAKKJJJIIIIIIIIIIIIIIIIIIIIIII:
:IIIIIIJJJJJJKKMAAAAAAAAAAAAAAAAAAAAAAAAAAAMLKJJJJJIIIIIIIIIIIIIIIIIIII:
:IIIIIJJJJJJKKLNAAAAAAAAAAAAAAAAAAAAAAAAAAAANLLKKJJJJIIIIIIIIIIIIIIIIII:
:IIIIJJJJJJJKKLMAAAAAAAAAAAAAAAAAAAAAAAAAAAONMLKKJJJIIIIIIIIIIIIIIIIIII:
:IIJJJJJJJJKKLMOAAAAAAAAAAAAAAAAAAAAAAAAAAAAKJJJJJJIIIIIIIIIIIIIIIIIIII:
:IJJJJJJJJJKKLMNAAAAAAAAAAAAAAAAAAAAAAAAAAAAOMLKKJJJJJJIIIIIIIIIIIIIIII:
:JJJJJJJKKKKLMAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAOMLKKJJJJJJJJIIIIIIIIIIIIII:
:JJJJJKKKKKLMNAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAMKKJJJJJJJJJIIIIIIIIIIIIII:
:JJJJKKKLLLLMAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAALKJJJJJJJJJJJJJJIIIIIIIII:
:JJJKKLMMMMMAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAALKJJJJJJJJJJJJJJJJJJJIIII:
:JJKKLAAAAANMNNNOOOOOOOOOOOOOOOOOOOOOOOONNNMMLLKKJJJJJJJJJJJJJJJJJJJJJJ:
:JJKKMAAAAANMMMMNNNNMMNNNMMMMMMMMMMMMMMMMMLLLLKKKJJJJJJJJJJJJJJJJJJJJJJ:
:JJKLMAAAAAONNNMMMMMMMMMMLLLLLLLLLLLLLLLLLKKKKKJJJJJJJJJJJJJJJJJJJJJJJJ:
:JJKLMAAAAAPOOOONMMMLLLLLLLLLLLLLLLKKKKKKKKKJJJJJJJJJJJJJJJJJJJJJJJJJJJ:
:JKKLMAAAAAAAAAAAANMMLLLLLLKKKKKKKKKKKKKKKKKJJJJJJJJJJJJJJJJJJJJJJJJJJJ:
:JKKLMAAAAAAAAAAAAOMMLLLKKKKKKKKKKKKKKKKKKKJJJJJJJJJJJJJJJJJJJJJJJJJJJJ:
:JKKKLAAAAAAAAAAAAPONMLLKKKKKKKKKKKKKKKKKKJJJJJJJJJJJJJJJJJJJJJJJJJJJJJ:
:JJKKLMMNNOPAAAAAAAAMLLKKKKKKKKKKKKKKKKKKKKJJJJJJJJJJJJJJJJJJJJJJJJJJJJ:
:JJKKKLLMMNOAAAAAAAANLLKKKKKKKKKKKKKKKKKKKKJJJJJJJJJJJJJJJJJJJJJJJJJJJJ:
:JJKKKKLLLMOAAAAAAAANLLKKKKKKKKKKKKKKKKKKKJJJJJJJJJJJJJJKKKKKJJJJJJJJJJ:
:JJJKKKKLLLNAAAAAAAAMLKKKKKKKKKKKKKKKJJJJJJJJJJJJJJJKKKKKKKKKKKKJJJJJJJ:
:JJJKKKKKLLMAAAAAAAAMLKKKKKKKKKKKKKKKJJJJJJJJJJJJJJJKKKKKKLLLKKKKJJJJJJ:
:JJJKKKKKKLLMMNNNNMMLLKKKKKKKKKKKKKKKKKKJJJJKKKKKKKKKKKKL   MLKKKKJJJJJ:
:JJJKKKKKKKLLLLMLLLLLLKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLM   LKKKKJJJJJ:
:JJJKKKKKKKKKLLLLLLKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLM   MLKKKKJJJJ:
:JJJJKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLM   MLLKKKKKKJ:
:JJJKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLN   NLLKKKKKKJ:
:JJKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLMNAAANLLLKKKKKKK:
:JKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLMNAAANMLLKKKKKKK:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLLMNAAANMLLLKKKKKK:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLLMMMOAAAOMMLLLKKKKK:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLMNNMNOAAAONMMLLLKKKK:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLMAAANNOAAAONNAMLLLKKK:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLMNAAAOOPAAAPNOANMLLLKK:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLLMNAAAOOPAAAPOOAANMLLKK:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLLMMMOAAAPOPAAAQPPAAOMLLLK:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLMMMMNOAAAPOPAAARRAAPNMLLK:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLM   NNOAAAPOQAAA   AAAOMLLK:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLM   NNOAAAPPQAAA   AAAOMMLL:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLN   ONOAAAPOQAAA   AAAONMLL:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLM   NNOAAAPOPAAA   AAAPNMLL:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLM   NNOAAAOOPAAA   AAAPNMLL:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLLMNNMMNAAAONOAAA   AAAONMLL:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLLLMMMMMNNNNNOPQ    AAAOMMLL:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLLLLLLLMMMMMMNNOP   AAAOMMLL:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLLLLLLLLLLLMMMMMNOP  AAONMLLL:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLLLLLLLLLLLLLLMMMNNO AONMMLLL:
:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLLLLLLLLLLLLLLLLMMMNNNNMMLLLL:
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:KKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKKLLLLLLLLLLLLLLLLLLLLLLLLLLMMMLLLLLLL:
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:LLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLL:
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```

MINTAB RESULTS—SCENARIO NO. 2
MINING STEP NO. 7

FIG. 3.85

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*****************************************
*  SIGZZ  POINT CONTOURS - MINING STEP  7 *
*****************************************


***** SYMBOL *****     ***** VALUE RANGE *****

        A             0.0          - 0.2500E 01
        B             0.2500E 01 - 0.5000E 01
        C             0.5000E 01 - 0.7500E 01
        D             0.7500E 01 - 0.1000E 02
        E             0.1000E 02 - 0.1250E 02
        F             0.1250E 02 - 0.1500E 02
        G             0.1500E 02 - 0.1750E 02
        H             0.1750E 02 - 0.2000E 02
        I             0.2000E 02 - 0.2250E 02
        J             0.2250E 02 - 0.2500E 02
        K             0.2500E 02 - 0.2750E 02
        L             0.2750E 02 - 0.3000E 02
        M             0.3000E 02 - 0.3250E 02
        N             0.3250E 02 - 0.3500E 02
        O             0.3500E 02 - 0.3750E 02
        P             0.3750E 02 - 0.4000E 02
        Q             0.4000E 02 - 0.4250E 02
        R             0.4250E 02 - 0.4500E 02

MINIMUM VALUE     =   0.473320E-03 MPA
MAXIMUM VALUE     =   0.447510E 02 MPA
CONTOUR INTERVAL =   0.250000E 01 MPA
```

MINTAB RESULTS–SCENARIO NO. 3
MINING STEP NO. 16

FIG. 3.86a

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---------------------------------------------------------------------
:IIIIIIIIIJJJJKKLAAAAAAAAAAAAAAAAAAAAAAAAAAALKJJJJIIIIIIIIIIIIIIIIIIII:
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:IIIIIJJJJJJKKLMNAAAAAAAAAAAAAAAAAAAAAAAAAAAANMLKKJJJJJIIIIIIIIIIIIIIII:
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:JJJKKLMMMMMNAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAAALKKJJJJJJJJJJJJJJJJJJJI:
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:JKKLMAAAAAAAAAAAAONNMMMMMNNO    ONCAANMMLLLLLLLKKKKKKKKJJJJJJJJJJJJJJJJJ:
:JKKLMAAAAAAAAAAAAPONNNNNNNO     OOOAAONMLLLLLLLLKKKKKKKKKKJJJJJJJJJJJJJJ:
:JKKKMAAAAAAAAAAAARQPPPOONNO     POOAAANMMLLLLLLLLLKKKKKKKKKKJJJJJJJJJJJJ:
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:JKKKKLLMMNPAAAAAAAA          POPAAAPOPAAAONNNNNMMMMMMLLLLKKKKKKKKKJJJJJJJJJ:
:JKKKKLLLMMOAAAAAAAAA         POPAAAPOPAAAONNAAANMNAAAMMLLLLLKKKKKKKKJJJJJJJ:
:JJKKKKKLLMNAAAAAAAAA         POPAAAPOPAAAPOOAAAONNAAANMMAMMLLLLLKKKKKKJJJJJJ:
:JJKKKKKKLLNAAAAAAAAA         POPAAAPOPAAAPOOAAAONOAAAONNAANMLLMMLLKKKKKJJJJJ:
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:LLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLL:
:LLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLLL:
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MINTAB RESULTS—SCENARIO NO. 3
MINING STEP NO. 16

FIG. 3.86b

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******************************************
*  SIGZZ  POINT CONTOURS - MINING STEP 16 *
******************************************



***** SYMBOL *****      ***** VALUE RANGE *****

          A             0.0          - 0.2500E 01
          B             0.2500E 01 - 0.5000E 01
          C             0.5000E 01 - 0.7500E 01
          D             0.7500E 01 - 0.1000E 02
          E             0.1000E 02 - 0.1250E 02
          F             0.1250E 02 - 0.1500E 02
          G             0.1500E 02 - 0.1750E 02
          H             0.1750E 02 - 0.2000E 02
          I             0.2000E 02 - 0.2250E 02
          J             0.2250E 02 - 0.2500E 02
          K             0.2500E 02 - 0.2750E 02
          L             0.2750E 02 - 0.3000E 02
          M             0.3000E 02 - 0.3250E 02
          N             0.3250E 02 - 0.3500E 02
          O             0.3500E 02 - 0.3750E 02
          P             0.3750E 02 - 0.4000E 02
          Q             0.4000E 02 - 0.4250E 02
          R             0.4250E 02 - 0.4500E 02

MINIMUM VALUE     =     0.907900E-02 MPA
MAXIMUM VALUE     =     0.426350E 02 MPA
CONTOUR INTERVAL  =     0.250000E 01 MPA
```

LIST OF APPENDICES

ROCK MASS CLASSIFICATION SYSTEMS A1 TO A25

ROCK MASS CLASSIFICATION (MATHEWS METHOD) B1 TO B17

APPENDIX A

ROCK MASS CLASSIFICATION SYSTEMS

A1 TO A9	**DATA COLLECTION AND ROCK MASS CLASSIFICATION**
A10 TO A15	**TABLES**
A16 TO A25	**FIGURES**

DATA COLLECTION & ROCK MASS CLASSIFICATION

1.0 GENERAL

Planning and design of openings in rock benefit from a number of empirical and semi-empirical rules. These serve to enable the designer to make an estimate of the expected stope dimensions and/or support conditions on the basis of a detailed description of the rock mass. The complete design procedure involves two steps. First, the quality of the rock mass is determined on the basis of a pre-defined classification system, and then the expected performance of the underground opening is predicted using empirically derived correlations with the rock quality factor.

The mining industry generally uses two schemes for classifying rock masses, Laubscher's MRMR and Barton's Q system. Although the systems differ, they rely on similar data to be collected. Once the data is collected, both systems are used to define the quality of the rock mass.

2.0 DATA COLLECTION FROM UNDERGROUND OPENINGS

2.1 Classification Parameters

The following parameters will be defined by data collection.

- Strength of Material;
- Joint Set Number;
- Joint Orientation;
- Joint Spacing;
- Block Shape;
- Joint Surface Alteration;
- Joint Surface Roughness;
- Ground Water;
- Stress Reduction Factor;
- Misc. Information (RQD, Avg. Block Vol.).

2.2 Data Collection Procedure

In order to fill out the data collection form (Figures 1A or 1B), the following procedure for data collection is recommended:

2.2.1 Select Area To Be Classified

Typically this will be the wall or back of a haulage, drill drift or drawpoint. The sampling site should cover an area of approximately 30 m^2 and should consist of predominantly one rock type.

2.2.2 Location

Record location and description of rock. A sketch is often useful for future reference.

2.2.3 Strength

The approximate strength (or hardness) of the rock should be determined using the methods described in Table 1.

2.2.4 Joint Set Number

From inspection, determine the number of joint sets present. If joint set orientations are reasonably consistent, as is typically the case, this is a straightforward task. Visual recognition of the number of sets should be accompanied by some system of numbering for identification purposes. For example, the most systematic and persistent set can be labelled "Set No. 1" and so on.

The number of sets recorded will tend to be a function of the size of area mapped, and should be interpreted accordingly. The spacing of individual sets will play an important role in this interpretation. For example, four sets recognized following a "conventional" survey of an area (using a stereonet contouring method) may include some sets with such wide spacing that these would be of little relevance to the stability of a short length of drift or stope.

2.2.5 Joint Set Orientation

For each of the joint sets defined, measure the mean discontinuity orientation (dip and dip direction). Three to five independent measurements of dip and dip direction should be taken and averaged. Dip should be recorded to the nearest 5 degrees, dip direction to the nearest 10 degrees.

2.2.6 Joint Spacing

For each joint set identified, the mean discontinuity spacing or frequency should be determined. This measurement can be rapidly performed as individual joint spacings are not of interest. Five or ten metres can be paced out or estimated with reasonable accuracy (i.e., within 10 per cent of the correct length). The observer should face in the direction of strike for each joint set that is to be counted and count perpendicular to the strike, thereby removing the angular correction factor. Mean spacing or discontinuity frequency over the sampled length can then be quickly determined, along with joint density (joints/m^3).

2.2.7 Block Shape

The exposure can be described by the following adjectives to give an impression of block size and shape (see Figure 2 for sketches):

(i) massive - few joints or very wide spacing;
(ii) blocky - approximately equidimensional;
(iii) tabular - one dimension considerably smaller than the other
(iv) columnar - one dimension considerably larger than the other two;
(v) irregular - wide variations of block size and shape;
(vi) crushed - heavily jointed to "sugar cube".

2.2.8 Joint Surface Alteration

For each joint set identified, the typical or average alteration conditions should be recorded using the table on the field data collection sheet (Figure 1A) or Table 2 as a guide. This is done in two steps. First, the separation between the joint surfaces is determined and secondly, the appropriate infill description in the table identified.

2.2.9 Joint Surface Roughness

For each joint set identified, the typical or average surface roughness should be recorded using the table on the field data collection sheet (Figure 1A) or Table 2 as a guide.

2.2.10 Stress Reduction Factor

Where appropriate, the stress reduction factor should be recorded using the table in the data collection sheet (Figure 1A) or Table 3 as a guide. If conditions observed are not found in the table, SRF should be set to one.

2.2.11 Other Information

Other information on the form includes average block volume and RQD.

RQD - When no core is available for determining RQD, it is possible to obtain an estimate from the mean discontinuity spacing. This relationship is shown graphically in Figure 4.

3.0 ROCK MASS CLASSIFICATION

3.1 Laubscher MRMR System

Laubscher (1984) defines a Mining Rock Mass Rating (MRMR) obtained from the summation of ratings for intact rock strength 0-20, RQD 0-15, joint spacing 0-25, and joint condition, 0-40, giving a total rating of 0-100. Descriptions and values associated with these basic ratings are given in Figure 3a. Once an overall rating is obtained a series of adjustments are done. These adjustments cater for the effects of:

- weathering;
- orientation of geologic structure in the back and walls of a stope or drift;
- blasting.

The adjustments allow the in-situ rock mass rating to be obtained.

The determination of an overall rating is relatively straightforward using the information given in Figures 3a, 3b, 3c and 3d. The following notes apply.

- Rock Quality Designation (RQD)

The RQD is a quantitative index based on a core recovery procedure in which the core recovery is determined incorporating only those pieces of hard, sound core which are 100 mm or greater in length. Shorter lengths are ignored. RQD is determined from the following expression:

$$RQD(\%) = 100 \times \frac{\text{length of core in pieces 100 mm or longer}}{\text{length of core run}}$$

Core of at least 50 mm in diameter should be used. If core of lesser or greater diameters is to be used, the nominal length of 100 mm should be altered to correspond to two times the core diameter. The International Society of Rock Mechanics recommends that the length of individual core pieces should be assessed along the centre line of the core, so that the discontinuities that happen to parallel the drill hole will not unduly penalize the RQD values of an otherwise massive rock mass. It is important to distinguish between mechanical and natural breaks found in the core. A mechanical break caused by handling should not adversely affect the RQD index which is a measure of the in-situ rock quality. The mechanical broken core segments should be approximated into a solid unit of core in order to arrive at a reliable RQD value.

RQD should be evaluated over an interval not less than 0.5 m. Typically 1.5 to 6 m intervals are used. When no core is available for determining RQD, it is possible to obtain an estimate from the mean discontinuity spacing. This estimate can be obtained from the graph presented in Figure 4. From Figure 3(a), a rating value for the measured RQD is determined.

- Intact Rock Strength (IRS)

The IRS value is the compressive strength of the rock without flaws such as joints and foliation. Unconfined compressive strength tests are usually used to determine this value. However, an estimate of strength derived from hardness is an acceptable starting point (see Table 1). From Figure 3(a), a rating value for the measured strength is determined.

- Joint Spacing

Provision is made in Figure 3(b) for obtaining the joint spacing rating for one, two or three joint systems. In the case of four or more joint sets then the three most frequently occurring joint sets are used (i.e., the closest spaced).

- Joint Condition and Ground Water

The rating is given in Figure 3(c). Consideration is given to discontinuity waviness (Parameter A), roughness (Parameter B), alteration (Parameter C), and infill (Parameter D), under four different ground water conditions.

The condition rating is then a combined percentage of the total possible of 40, that is,

$$\text{Joint condition rating} = 40 \times \frac{A}{100} \times \frac{B}{100} \times \frac{C}{100} \times \frac{D}{100}$$

- Adjustments to MRMR
  - Weathering: for rocks that deteriorate readily during the life of an opening, adjust RQD rating by 95 per cent, UCS rating by 96 per cent, and joint condition by 82 per cent, giving up to 75 per cent adjustment to MRMR.
  - Joint Orientation: the orientation of joint sets will define those rock blocks that may kinematically slide or fall out from an excavation surface. The adjustments, simplified and slightly modified from Laubscher's original proposal are shown in Figure 3d. These are made to the joint spacing rating.
  - Blasting: the excavation method significantly affects the opening periphery. The recommended classification adjustments are as follows:

Boring	100%
Smooth wall blasting	97%
Conventional blasting (good)	94%
Conventional blasting (poor)	80%

3.2 Barton Q System

A similar concept to the MRMR is used in the NGI System (Barton, et al., 1974). In this scheme the structure of the rock mass, the frictional characteristics of fractures, and stresses surrounding the opening are assessed and combined to give the Tunnelling Quality Index (Q).

The system has the form:

$$Q = \frac{RQD}{J_n} \cdot \frac{J_r}{J_a} \cdot \frac{J_w}{SRF}$$

Where:

RQD = denotes the rock quality designation
J_n = the joint set number
J_r = the joint roughness number
J_a = the joint alteration number
J_w = the joint water reduction factor
SRF = the stress reduction factor

The physical significance of the Index is that the term RQD/J_n represents the average block size of the mass; J_r/J_a the average total shearing strength of the joints; J_w both the relative potential head of water on the support and the relative potential of the water eroding the joint infilling; and SRF, both the structurally controlled possibility of dislodgement of blocks of ground and the concentrated stress related confinement, fracturing, squeezing and swelling of the excavation walls.

- Rock Quality Designation (RQD)

The method of determining RQD is discussed in the section on the MRMR classification. The RQD value is used directly in the Q classification system.

- Joint Set Number (J_n)

A rating for the number of joint sets present is determined from the appropriate chart in Table 2.

- Joint Roughness Number (J_r)

A rating for average joint roughness is determined from the appropriate chart on Table 2. Where joint roughness varies the "worst case" surface on a major joint set should be utilized.

- Joint Alteration Number (J_a)

Joint alteration rating is determined from the appropriate chart in Table 2. Both the separation between joint surfaces and joint infilling are considered. Where joint alteration varies, a value corresponding to the average of the "worst case" alteration on a major joint set should be used.

- Joint Water Reduction (J_w)

Due to the extensive and interconnected geometry of underground openings, and the fractured nature of the rock mass immediately adjacent to these openings, it is usually assumed that the rock mass is in a drained condition ($J_w = 1$).

- Stress Reduction Factor (SRF)

The stress reduction factor is determined from the appropriate chart in Table 2. Alternatively the SRF can be determined from relationships relating rock strength and in situ stress, developed by Kirsten (1983). Values can be determined from Table 3.

A significant background of experience has been built up with the NGI system. Hoek and Brown (1980) use the NGI system for evaluation of rock mass strength and Mathews et al (1981) have reported success in using it for assessing open stope design.

In order to relate the index value (Q) to the behaviour and support requirements of an underground excavation an additional quantity called the equivalent dimension (D_e) is used. This dimension is obtained by dividing the span, diameter, or wall height of the excavation by a quantity called the excavation support ratio (ESR).

Hence:

$$D_e = \frac{\text{Excavation span, diameter or height (m)}}{\text{Excavation Support Ratio}}$$

The excavation support ratio is related to the use for which the excavation is intended and the extent to which some degree of instability is acceptable. Barton (1976) gives the following suggested values for ESR:

Excavation Category		ESR
A.	Temporary mine openings.	3 - 5
B.	Permanent mine openings, water tunnels for hydro power (excluding high pressure penstocks) pilot tunnels, drifts and headings for large excavations.	1.6
C.	Storage rooms, water treatment plants, minor road and railway tunnels, surge chambers, access tunnels.	1.3
D.	Power stations, major road and railway tunnels, civil defence chambers, portals, intersections.	1.0
E.	Underground nuclear power stations, railway stations, sports and public facilities, factories.	0.8

The ESR is roughly analogous to the inverse of the factor of safety used in the design of rock slopes.

The relationship between the Tunnelling Quality Index Q and the Equivalent Dimension D_e of an excavation which will stand unsupported is illustrated in Figure 5.

4.0 BLOCK GEOMETRY

A useful adjunct to the various classification systems previously discussed is to obtain a measure of block shape. Chappell (1987) has shown that block shape and block volume is important in assessing the re-distribution of stress that takes place when an excavation surface deforms. The initial step in describing block geometry is to determine the average joint spacing or frequency for each of the major joint sets identified. The process of data reduction used to describe joint spacing is outlined below.

The mean spacing Sp for individual joint sets can be obtained by:

$$Sp = 1/\lambda$$

where λ is the mean frequency. Thus, at a specific sampling location, a line of known length should be drawn approximately perpendicular to the strike of the joint set under review. By counting the number of fractures that intersect the line, the fracture frequency and thus average spacing can be estimated.

If it is assumed that joint length is greater than joint spacing, the block size and shape can be defined by either joint spacing or fracture frequency.

For average block size, side lengths L, I and S are defined by mean joint spacing, that is:

L = mean spacing joint set Y = $1/\lambda y$
I = mean spacing joint set X = $1/\lambda x$
S = mean spacing joint set Z = $1/\lambda z$

If four or more joint sets exist, it is assumed that the block is defined by those three sets with the least mean spacing (i.e., the most frequently occurring joints).

Once the edge lengths are obtained, block shape is classified (see Figure 6). The shape is also an indirect measure of anisotropy.

The proposed shape classification system is based on techniques used in sedimentology for the description of grain shape. Folk (1968) defines form as a measure of the relation between the three dimensions of an object. Thus particles (or rock blocks) may be classed qualitatively as compact (or equidimensional) elongated (or rodlike) and platy, with several intermediate categories. The so-called 'form triangle' is shown on Figure 6. The triangle is made up of three axes, namely the ratio of S/L, (L-I)/(L-S) and S^2/LI where S, I, and L are the measured edge lengths of the block. Any two of these axes can be used to define block shape.

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TABLE 1 - FIELD ESTIMATION OF ROCK HARDNESS

Grade	Description	Field Identification	Approximate Range of Uniaxial Compressive Strength (MPa)
S1	Very soft clay	Easily penetrated several inches by fist	<0.025
S2	Soft clay	Easily penetrated several inches by thumb	0.025-0.05
S3	Firm clay	Can be penetrated several inches by thumb with moderate effort	0.05-0.10
S4	Stiff clay	Readily indented by thumb but penetrated only with great effort	0.10-0.25
S5	Very stiff clay	Readily indented by thumbnail	0.25-0.50
S6	Hard clay	Indented with difficulty by thumbnail	>0.50
R0	Extremely weak rock	Indented by thumbnail	0.25-1.0
R1	Very weak rock	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife	1.0-5.0
R2	Weak rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer	5.0-25
R3	Medium strong rock	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer	25-50
R4	Strong rock	Specimen requires more than one blow of geological hammer to fracture it	50-100
R5	Very strong rock	Specimen requires many blows of geological hammer to fracture it	100-250
R6	Extremely strong rock	Specimen can only be chipped with geological hammer	>250

Reference: Brown, 1981

TABLE 2 - CLASSIFICATION OF INDIVIDUAL PARAMETERS USED IN THE NGI TUNNELLING QUALITY INDEX

Description	*Value*	*Notes*
1. ROCK QUALITY DESIGNATION	RQD	
A. Very poor	0 - 25	1. Where RQD is reported or measured as $\leq$ 10 (including 0), a nominal value of 10 is used to evaluate Q.
B. Poor	25 - 50	
C. Fair	50 - 75	
D. Good	75 - 90	2. RQD intervals of 5, i.e. 100, 95, 90 etc are sufficiently accurate.
E. Excellent	90 - 100	
2. JOINT SET NUMBER	J_n	
A. Massive, no or few joints	0.5 - 1.0	
B. One joint set	2	
C. One joint set plus random	3	
D. Two joint sets	4	
E. Two joint sets plus random	6	
F. Three joint sets	9	1. For intersections use $(3.0 \times J_n)$
G. Three joint sets plus random	12	
H. Four or more joint sets, random, heavily jointed 'sugar cube', etc	15	2. For portals use $(2.0 \times J_n)$
J. Crushed rock, earthlike	20	
3. JOINT ROUGHNESS NUMBER	J_r	
a. Rock wall contact and		
b. Rock wall contact before 10 cms shear.		
A. Discontinuous joints	4	
B. Rough or irregular, undulating	3	
C. Smooth, undulating	2	
D. Slickensided, undulating	1.5	1. Add 1.0 if the mean spacing of the relevant joint set is greater than 3m.
E. Rough or irregular, planar	1.5	
F. Smooth, planar	1.0	2. J_r = 0.5 can be used for planar, slickensided joints having lineations, provided the lineations are orientated for minimum strength.
G. Slickensided, planar	0.5	
c. No rock wall contact when sheared.		
H. Zone containing clay minerals thick enough to prevent rock wall contact.	1.0	
J. Sandy, gravelly or crushed zone thick enough to prevent rock wall contact.	1.0	
4. JOINT ALTERATION NUMBER	J_a	ϕ_r (approx.)
a. Rock wall contact.		
A. Tightly healed, hard, non-softening, impermeable filling	0.75	

TABLE 2 - CLASSIFICATION OF INDIVIDUAL PARAMETERS USED IN THE NGI TUNNELLING QUALITY INDEX (cont'd)

	J_a	ϕ_r (approx.)	
B. Unaltered joint walls, surface staining only	1.0	(25° - 35°)	
C. Slightly altered joint walls non-softening mineral coatings, sandy particles, clay-free disintegrated rock, etc	2.0	(25° - 30°)	1. Values of ϕ_r, the residual friction angle, are intended as an approximate guide to the mineralogical properties of the alteration products, if present.
D. Silty-, or sandy-clay coatings, small clay-fraction (non-softening)	3.0	(20° - 25°)	
E. Softening or low friction clay mineral coatings, i.e. kaolinite, mica. Also chlorite, talc, gypsum and graphite etc., and small quantities of swelling clays. (Discontinuous coatings, 1-2mm or less in thickness)	4.0	(8° - 16°)	
b. Rock wall contact before 10 cms shear.			
F. Sandy particles, clay-free disintegrated rock etc	4.0	(25° - 30°)	
G. Strongly over-consolidated, non-softening clay mineral fillings (continuous, < 5mm thick)	6.0	(16° - 24°)	
H. Medium or low over-consolidation, softening, clay mineral fillings, (continuous, < 5mm thick)	8.0	(12° - 16°)	
J. Swelling clay fillings, i.e. montmorillonite (continuous, < 5 mm thick). Values of J_a depend on percent of swelling clay-size particles, and access to water	8.0 - 12.0	(6° - 12°)	
c. No rock wall contact when sheared.			
K. Zones or bands of disintegrated L. or crushed rock and clay (see M. G,H and J for clay conditions)	6.0 8.0 8.0 - 12.0	(6° - 24°)	
N. Zones or bands of silty- or sandy clay, small clay fraction, (non-softening)	5.0		
Q. Thick, continuous zones or P. bands of clay (see G, H and R. J for clay conditions)	10.0 - 13.0 13.0 - 20.0	(6° - 24°)	

5. JOINT WATER REDUCTION FACTOR	J_w	approx. water pressure (Kgf/cm²)	
A. Dry excavations or minor inflow, i.e. < 5 lit/min. locally	1.0	< 1.0	
B. Medium inflow or pressure, occasional outwash of joint fillings	0.66	1.0 - 2.5	
C. Large inflow or high pressure in competent rock with unfilled joints	0.5	2.5 - 10.0	1. Factors C to F are crude estimates. Increase J_w if drainage measures are installed.
D. Large inflow or high pressure , considerable outwash of fillings	0.33	2.5 - 10.0	
E. Exceptionally high inflow or pressure at blasting, decaying with time	0.2 - 0.1	> 10	2. Special problems caused by ice formation are not considered.
F. Exceptionally high inflow or pressure continuing without decay	0.1 - 0.05	> 10	

TABLE 2 - CLASSIFICATION OF INDIVIDUAL PARAMETERS USED IN THE NGI TUNNELLING QUALITY INDEX (cont'd)

6. STRESS REDUCTION FACTOR

a. Weakness zones intersecting excavation, which may cause loosening of rock mass when tunnel is excavated.

	SRF
A. Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth)	10.0
B. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth < 50m)	5.0
C. Single weakness zones containing clay, or chemically disintegrated rock (excavation depth > 50m)	2.5
D. Multiple shear zones in competent rock (clay free), loose surrounding rock (any depth)	7.5
E. Single shear zones in competent rock (clay free), (depth of excavation < 50m)	5.0
F. Single shear zones in competent rock (clay free), (depth of excavation > 50m)	2.5
G. Loose open joints, heavily jointed or 'sugar cube' (any depth)	5.0

1. Reduce these values of SRF by 25 - 50% if the relevent shear zones only influence but do not intersect the excavation.

b. Competant rock, rock stress problems

	σ_c/σ_1	σ_t/σ_1	SRF
H. Low stress, near surface	>200	>13	2.5
J. Medium stress	200-10	13-0.66	1.0
K. High stress, very tight structure (usually favourable to stability, may be unfavourable for wall stability)	10-5	0.66-0.33	0.5-2
L. Mild rock burst (massive rock)	5-2.5	0.33-0.16	5-10
M. Heavy rock burst (massive rock)	<2.5	<0.16	10-20

2. For strongly anisotropic virgin stress field (if measured) : when $5 \leq \sigma_1/\sigma_3 \leq 10$, reduce σ_c to $0.8\sigma_c$ and σ_t to $0.8\sigma_t$. When $\sigma_1/\sigma_3 > 10$, reduce σ_c and σ_t to $0.6\sigma_c$ and $0.6\sigma_t$, where σ_c = unconfined compressive strength, and σ_t = tensile strength (point load) and σ_1 and σ_3 are the major and minor principal stresses.
3. Few case records available where depth of crown below surface is less than span width. Suggest SRF increase from 2.5 to 5 for such cases (see H).

c. Squeezing rock, plastic flow of incompetent rock under the influence of high rock pressure

	SRF
N. Mild squeezing rock pressure	5-10
O. Heavy squeezing rock pressure	10-20

d. Swelling rock, chemical swelling activity depending upon presence of water

P. Mild swelling rock pressure	5-10
R. Heavy swelling rock pressure	10-20

ADDITIONAL NOTES ON THE USE OF THESE TABLES

When making estimates of the rock mass quality (Q) the following guidelines should be followed, in addition to the notes listed in the tables:

1. When borehole core is unavailable, RQD can be estimated from the number of joints per unit volume, in which the number of joints per metre for each joint set are added. A simple relation can be used to convert this number to RQD for the case of clay free rock masses :
   $RQD = 115 - 3.3J_v$ (approx.) where J_v = total number of joints per m^3
   ($RQD = 100$ for $J_v < 4.5$)
2. The parameter J_n representing the number of joint sets will often be affected by foliation, schistosity, slaty cleavage or bedding etc. If strongly developed these parallel "joints" should obviously be counted as a complete joint set. However, if there are few "joints" visible, or only occasional breaks in the core due to these features, then it will be more appropriate to count them as "random joints" when evaluating J_n.
3. The parameters J_r and J_a (representing shear strength) should be relevant to the *weakest significant joint set or clay filled discontinuity* in the given zone. However, if the joint set or discontinuity with the minimum value of (J_r/J_a) is favourably oriented for stability, then a second, less favourably oriented joint set or discontinuity may sometimes be more significant, and its higher value of J_r/J_a should be used when evaluating Q . *The value of J_r/J_a should in fact relate to the surface most likely to allow failure to initiate.*
4. When a rock mass contains clay, the factor SRF appropriate to *loosening loads* should be evaluated. In such cases the strength of the intact rock is of little interest. However, when jointing is minimal and clay is completely absent the strength of the intact rock may

TABLE 2 - CLASSIFICATION OF INDIVIDUAL PARAMETERS USED IN THE NGI TUNNELLING QUALITY INDEX

become the weakest link, and the stability will then depend on the ratio rock-stress/rock-strength. A strongly anisotropic stress field is unfavourable for stability and is roughly accounted for as in note 2 in the table for stress reduction factor evaluation.

5. The compressive and tensile strengths (σ_c and σ_t) of the intact rock should be evaluated in the saturated condition if this is appropriate to present or future in situ conditions. A very conservative estimate of strength should be made for those rocks that deteriorate when exposed to moist or saturated conditions.

TABLE 3 - STRESS REDUCTION FACTOR (SRF)

Material Type	Description of Intact Ground	SRF for Rock Mass Predominated, Fracturing, Squeezing or Swelling																	Single Weakness Zone or Joint Infill Predominated Confinement or Squeezing	
		Unconfined Compressive Strength (MPa)	Head of Rock, N, (a)																0	SRF
			2	4	6	8	10	20	40	60	80	100	200	300	400	600	800	1000	0.001	17.5
Granular/ Cohesion Soil	Very Loose/Very Soft	0.035	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	0.002	14.0
	Loose/Soft	0.069	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	0.003	10.3
	Medium Dense/Firm	0.14	11.4	20	20	20	20	20	20	20	20	20	20	20	20	20	20	20	0.01	0.2
	Coarse/Stiff	0.28	4.6	11.4	19.5	20	20	20	20	20	20	20	20	20	20	20	20	20	0.04	5.2
	Very Dense/Very Stiff	0.55	1.9	4.7	8.0	11.7	15.7	20	20	20	20	20	20	20	20	20	20	20	0.1	3.9
Rock	Very Soft Rock	1.1	0.8	1.9	3.2	4.7	6.3	15.7	20	20	20	20	20	20	20	20	20	20	0.4	2.4
	Medium Soft Rock	2.2	0.5	0.8	1.3	1.9	2.3	6.3	15.7	20	20	20	20	20	20	20	20	20	1	1.0
	Soft Rock	4.4	0.7	0.5	0.6	0.8	1.1	2.3	6.3	10.7	15.7	20	20	20	20	20	20	20	2	1.4
	Medium Hard Rock	8.8	1.5	0.7	0.5	0.5	0.5	1.1	2.5	4.3	6.3	8.4	20	20	20	20	20	20	3	1.3
	Hard Rock	17.7	3.9	1.5	0.9	0.7	0.6	0.5	1.1	1.7	2.9	3.4	8.4	14.8	20	20	20	20	4	1.1
	Very Hard Rock	35.4	9	3.9	2.2	1.9	1.1	0.6	0.9	0.8	1.1	1.4	3.4	5.7	8.4	14.3	20	20	6	1.0
	Very Very Hard Rock	70.7	5	5	5	3.8	2.8	1.1	0.6	0.3	0.5	0.6	1.4	2.3	3.4	5.7	8.4	11.3	10	0.9
	Extremely Hard Rock	141.3	5	5	5	5	5	2.8	1.1	0.7	0.6	0.5	0.6	1.0	1.4	2.3	3.4	4.5	20	0.7
	Exceptionally Hard Rock	282.4	9	5	5	5	5	5	2.8	1.6	1.1	0.9	0.5	0.5	0.6	1.0	1.4	1.8	30	0.4

ROCK MASS CLASSIFICATION DATA SHEET

Figure **1a**

LOCATION:
Level ____________ Face ____________ Distance ____________

DATE

Rock Type	Strength	Sketch

Joint Data	Set 1	Set 2	Set 3	Set 4
Dip Direct/Dip Angle				
Average Spacing (ft)				
Estimated No. of Joints/m^3				

Q =

MRMR =

Avg. Vol. =

Shape

RQD

JOINT ALTERNATION NUMBER (Ja)

Description of gouge	Joint alteration number (Ja) for joint separation (mm)		
	<1.0[1]	1.0-5.0[2]	>5.0[3]
Tightly healed, hard, nonsoftening impermeable filling	0.75	-	-
Unaltered joint walls, surface staining only	1.0	-	-
Slightly altered, non-softening, non-cohesive rock mineral or crushed rock filling	2.0	4.0	6.0
Non-softening, slightly clayey non-cohesive filling	3.0	6.0	10.0
Softening or low friction clay mineral coatings and small quantities of swelling clays	4.0	8.0[5]	13.0
Shattered or micro-shattered (swelling) clay gouge, with or without crushed rock	5.0	10.0[4]	18.0

JOINT ROUGHNESS NUMBER (Jr)

Joint Separation	Condition of Joint	Joint Roughness Number (Jr)
Joints/fissures tight or losing during excavation	Discontinuous joint/fissures	4.0
	Rough or irregular, indulating	3.0
	Smooth indulating	2.0
	Rough or irregular, planar	1.5
	Smooth planar	1.0
	Slickensided planar	0.5
Joints/fissures open and remain open during excavation	Joints/fissures either open or containing relatively soft gouge or sufficient thickness to prevent joint/fissure wall contact upon excavation.	1.0
	Shattered or micro-shattered clays	1.0

JOINT SET NUMBER (Jn)

Number of joint sets	Joint set number (Jn)
Intact, no or few joint/fissures	0.5 - 1.0
One joint/fissure set	2
One joint/fissure set plus random	3
Two joint/fissure sets	4
Two joint/fissure sets plus random	6
Three joint/fissure sets	9
Three joint/fissure sets plus random	12
Four or more joint/fissure sets, random, heavily jointed, sugar cube, etc.	15
Multiple joint/fissure sets (crushed rock, earth-like)	20

STRESS REDUCTION FACTOR (SRF)

Weakness zones intersecting pillar.	10.0
Multiple occurrences of weakness zones containing clay or chemically disintegrated rock, very loose surrounding rock (any depth).	2.5
Single weakness zones containing clay, or chemically disintegrated rock.	2.5
Multiple shear zones in competent rock (clay free), loose surrounding rock.	7.5
Single shear zones in competent rock (clay free).	2.5
Loose open joints, heavily jointed or 'sugar cube'.	5.0

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

ROCK MASS CLASSIFICATION

Figure **1b**

Site:	Collected by:	Date:

Description of sample area:

Rock type	Strength	Sketch

Structural Data	Set 1	Set 2	Set 3	Set 4
Dip/Dip Direction				
Average Spacing (m)				
Estimated No. of Joints/m^3				

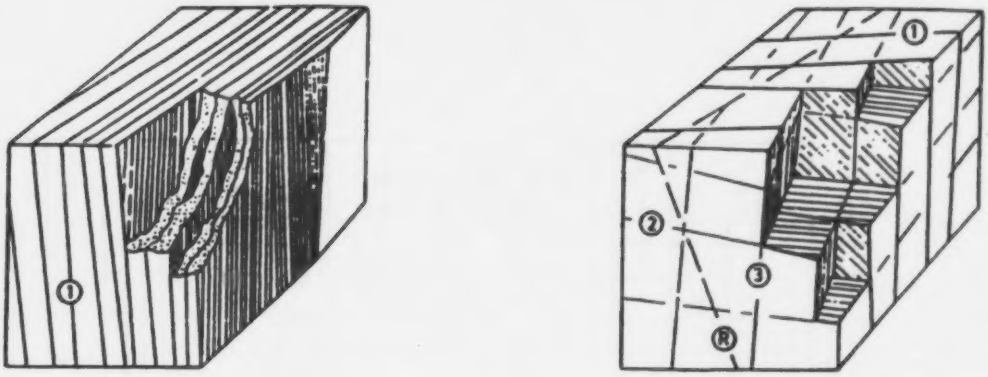
MRMR Classification		Q Classification	
Strength (IRS)		RQD	
RQD		Joint Set No.	
Joint Spacing		Joint Roughness No.	
Joint Condition		Joint Alteration	
Adjustments		Joint Water Reduction	
		Stress Reduction	
MRMR		Tunnelling Quality, Q	

Other Notes:

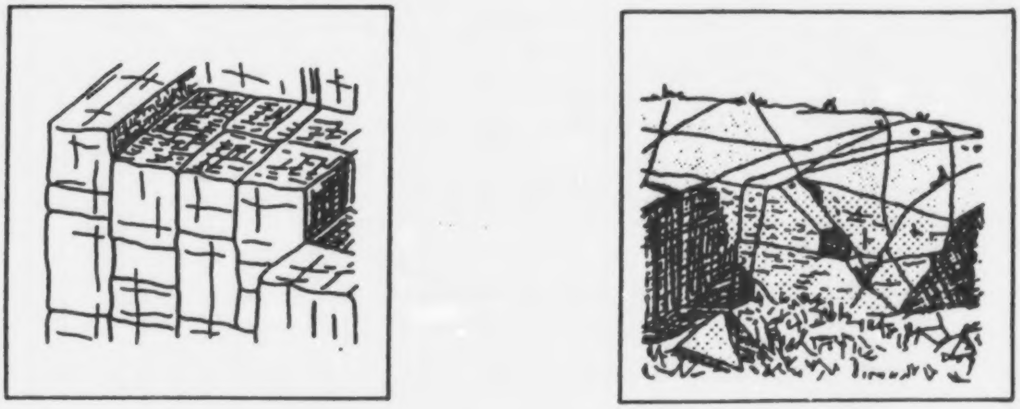
PROJECT NO. .... DRAWN .... - .... REVIEWED .... DATE ....

EXAMPLES THAT DEMONSTRATE THE EFFECT OF THE NUMBER OF JOINT SETS ON THE MECHANICAL BEHAVIOUR AND APPEARANCE OF A ROCK MASS

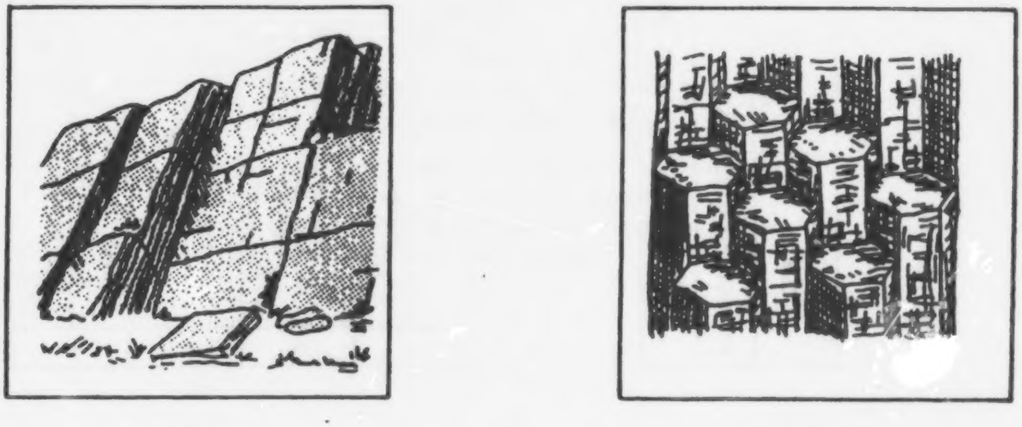
Figure **2**



One Joint Set *Three Joint Sets*



Blocky *Irregular*



Tabular *Columnar*

PROJECT NO.......... DRAWN - REVIEWED .... DATE ....

PROJECT NO.………… DRAWN - REVIEWED ……… DATE …………

MODIFIED GEOMECHANICS CLASSIFICATION SCHEME (after Laubscher, 1984)

Figure **3a**

A. MEANING OF THE RATINGS

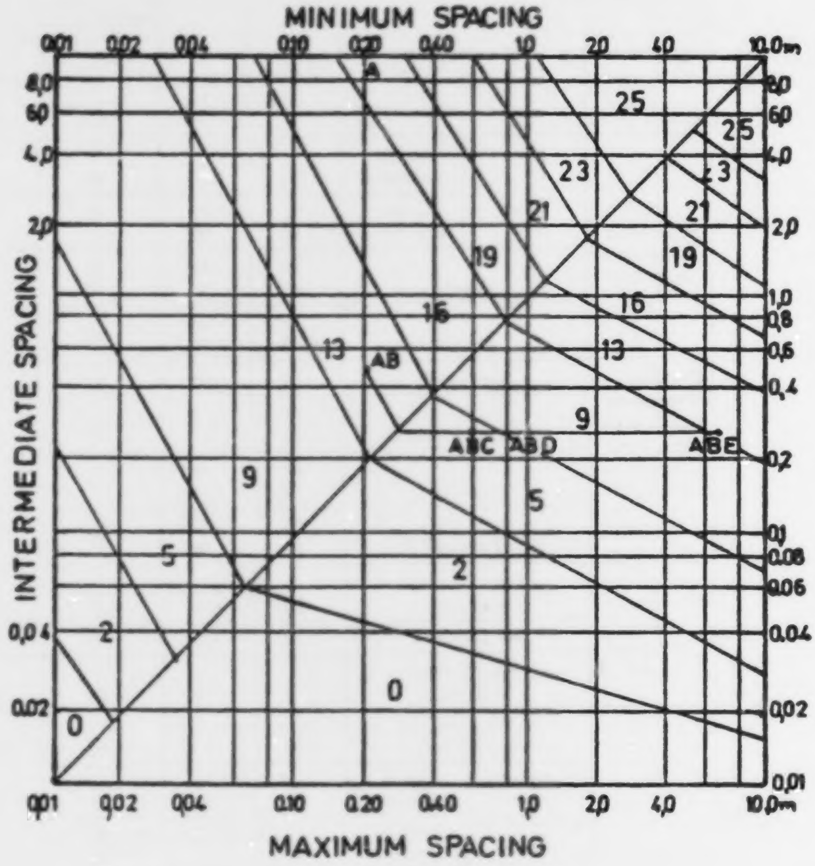
CLASS	1		2		3		4		5	
	A	B	A	B	A	B	A	B	A	B
RATING(Σ1-4 OF B)	100 - 81		80 - 61		60 — 41		40 - 21		20 - 0	
DESCRIPTION	VERY GOOD		GOOD		FAIR		POOR		VERY POOR	

B. BASIS OF THE CLASSIFICATION

1	ROD %	100 — 97		96-84	83 -71	70 - 58	55 - 44	43 -31	30 - 17	16 - 4	3 — 0	
	RATING (= ROD % x 15/ 100)	15		14	12	10	8	6	4	2	0	
2	IRS (MPa)	>185	184-165	164 145	144 125	124-105	104-85	84-65	64-45	44 - 25	24 -5	4-0
	RATING (= 0,1×MPa)	20	18	16	14	12	10	8	6	4	2	0
3	JOINT SPACING	REFER TO FIGURE I-4b										
	RATING	25 ←——→ 0										
4	JOINT CONDITION INCL GROUND WATER	REFER TO FIGURE I-4c										
	RATING ($40 \times A \times B \times C \times D/10^8$)	40 ←——→ 0										

ORIENTATION ADJUSTMENT (modified from Laubscher, 1984)

Figure **3b**



EXAMPLES:
SPACING A=0.2m, B=0.45m, C=0.5m, D=1.0m, E=7m
RATINGS A=19, AB=13, ABC=5, ABD=9, ABE=13

Locate point corresponding to minimum and intermediate spaced joints (point AB) locate point ABC or ABD by constructing parallel lines read off spacing value.

ADJUSTMENTS FOR JOINT CONDITION AND GROUNDWATER

Figure **3c**

Parameter	Description		Dry condition	Wet Conditions		
				Moist	Moderate pressure 25—125 l/min	Severe pressure > 125 l/min
A Joint expression (large scale irregularities)	Wavy	Multi-directional	100	100	95	90
		Uni-directional	95 90	95 90	90 85	80 75
	Curved		89 80	85 75	80 70	70 60
	Straight		79 70	74 65	60	40
B Joint expression (smale scale irregularities or roughness)	Very rough		100	100	95	90
	Striated or rough		99 85	99 85	80	70
	Smooth		84 60	80 55	60	50
	Polished		59 50	50 40	30	20
C Joint Wall alteration zone	Stronger than wall rock		100	100	100	100
	No alteration		100	100	100	100
	Weaker than wall rock		75	70	65	60
D Joint filling	No fill — surface staining only		100	100	100	100
	Non-softening and sheared material (clay or talc free)	Coarse sheared	95	90	70	50
		Medium sheared	90	85	65	45
		Fine sheared	85	80	60	40
	Soft sheared material (eg talc)	Coarse sheared	70	65	40	20
		Medium sheared	65	60	35	15
		Fine sheared	60	55	30	10
	Gouge thickness <amplitude of irregularity		40	30	10	
	Gouge thickness >amplitude of irregularity		20	10	Flowing material 5	

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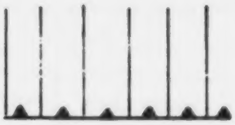
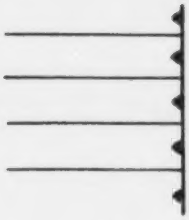
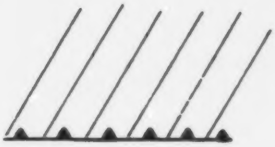
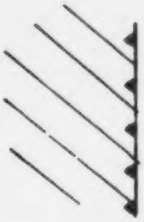
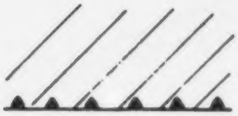
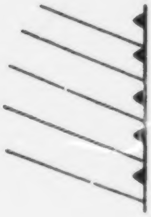
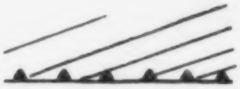
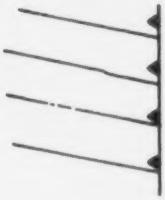
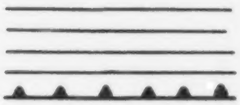
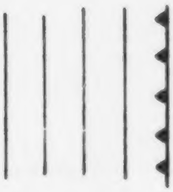
ORIENTATION ADJUSTMENT
(MODIFIED FROM LAUBSCHER, 1984)

Figure **3d**

Dip of Structure	Back/Roof	Adjustment	Wall	Adjustment
90°		100%		100%
60°		90%		70%
45°		75%		80%
20°		70%		90%
0		80%		80%

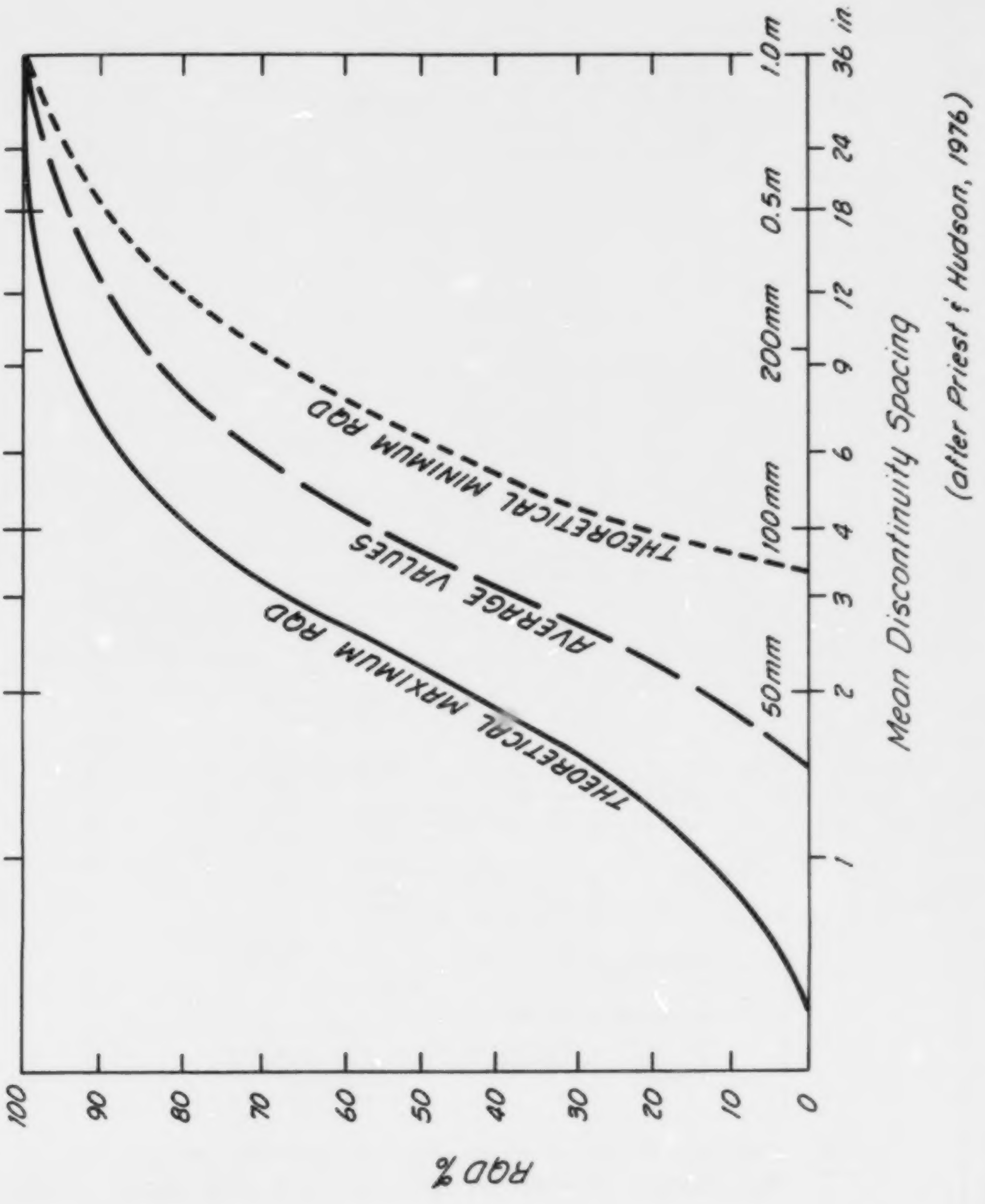
Note: Structure refers to plunge of specific discontinuity set or the plunge of intersection between two major joints sets.

PROJECT NO. .... DRAWN - .... REVIEWED .... DATE ....

RQD VERSUS MEAN DISCONTINUITY SPACING

Figure **4**

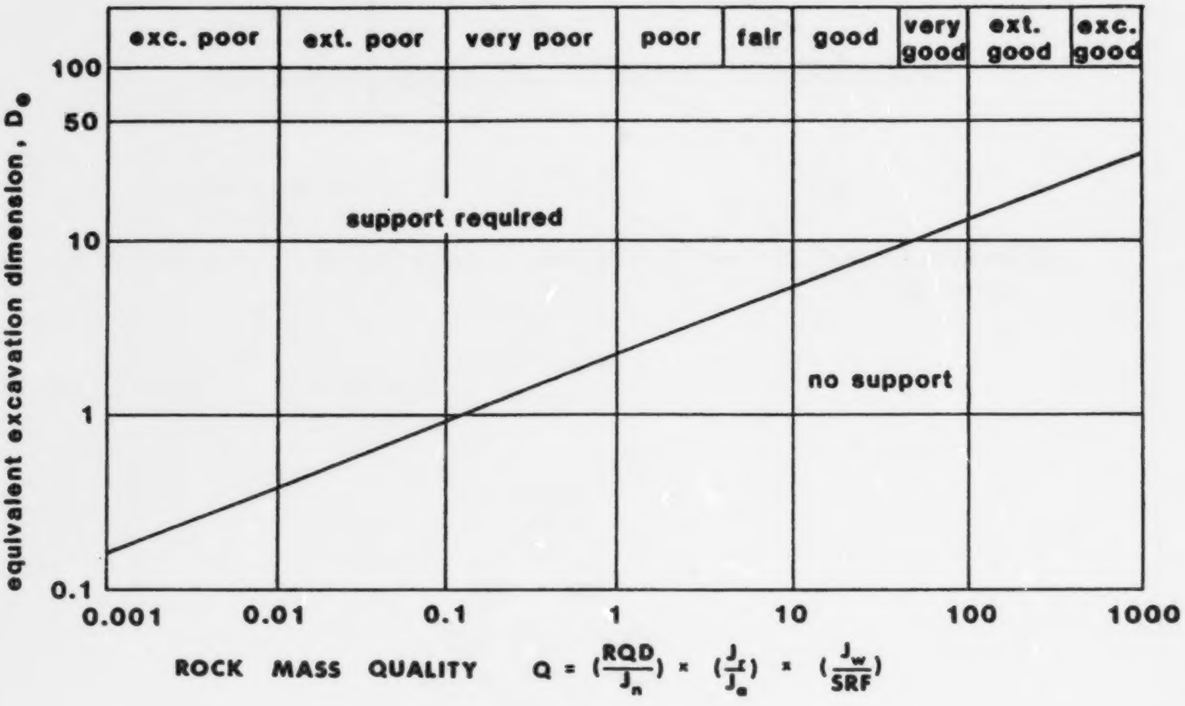
PROJECT NO. .... DRAWN - .... REVIEWED .... DATE ....



(after Priest & Hudson, 1976)

RELATING SPAN AND NGI

Figure **5**



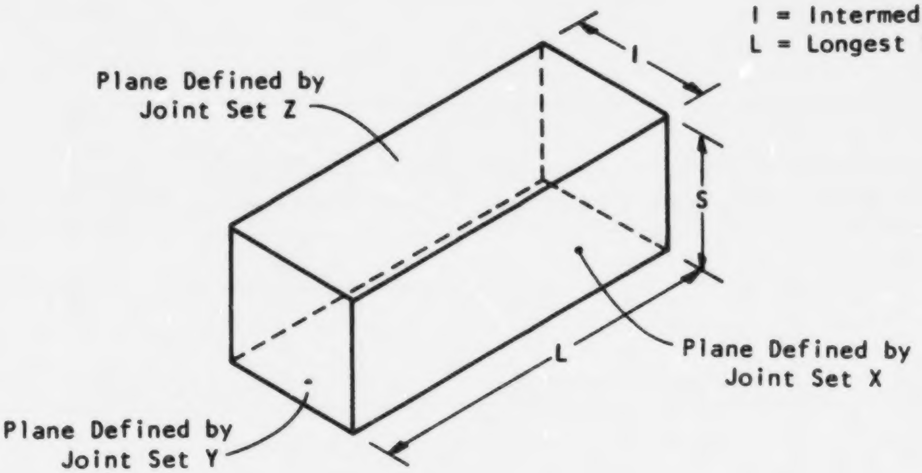
from Barton et al, 1974

PROJECT NO. .... DRAWN .... REVIEWED .... DATE ....

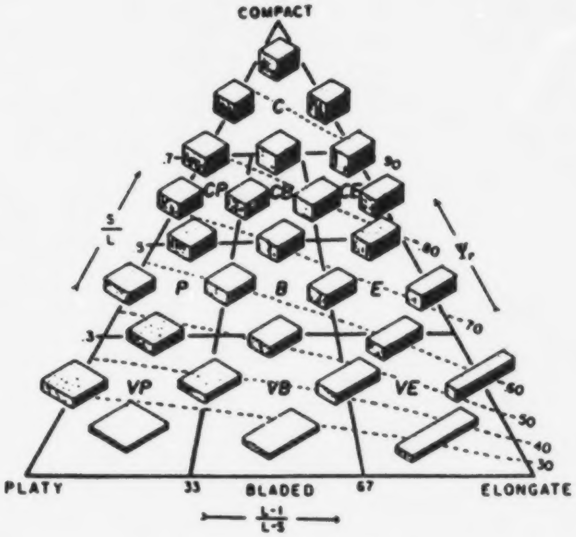
PROPOSED CLASSIFICATION OF BLOCK SHAPE (after Folk, 1965)

Figure **6&7**

S = Shortest Side Length
I = Intermediate Side Length
L = Longest Side Length


Plane Defined by Joint Set Z
I
S
L
Plane Defined by Joint Set X
Plane Defined by Joint Set Y

Block Shape


COMPACT
C
.7
CP
CB
CE
90
$\frac{S}{L}$
.5
80
Ψ_r
P
B
E
.3
70
VP
VB
VE
60
50
40
30
PLATY
33
BLADED
67
ELONGATE
$\frac{L-I}{L-S}$

C = Compact (cube)
CP = Compact Platy
P = Platy
VP = Very Platy

VB = Very Bladed
B = Bladed
CB = Compact Bladed
CE = Compact Elongate

E = Elongate
VE = Very Elongate

APPENDIX B

ROCK MASS CLASSIFICATION (MATHEWS METHOD)

B1 TO B9	**ROCK MASS CLASSIFICATION (MATHEWS METHOD)**
B10 TO B17	**FIGURES**

ROCK MASS CLASSIFICATION (MATHEWS METHOD)

1.0 INTRODUCTION

Golder Associates (1981) developed an empirical relationship between depth of mining, rock mass quality, and stope span. Geotechnical parameters, summarized in the form of a stability number, are plotted against an excavation size factor to assess the stability of surfaces bounding an opening. The stability number accounts for rock mass quality, the state of stress and the orientation of the exposed surfaces. The excavation factor accounts for the shape and size of the opening. On this basis, a high stability number and low stope factor should reflect stable exposures while the converse should reflect instability. It is necessary to assess the stability of each exposure in turn; roof, hangingwall, horizontal pillars and vertical pillars, etc.

2.0 CALCULATION OF STABILITY NUMBER (N)

The stability number (N) is expressed as:

$$N = Q' \times A \times B \times C$$

Q'	is the modified NGI Rock Mass rating (Barton et al, 1974):
A	is a stress factor
B	is a rock defect orientation factor
C	is a design surface orientation factor

Within the stability number Golder Associates (1981) have attempted to account for three types of failure. These are:

- Structurally controlled failures caused by opening and movement along unfavourably oriented structure (usually zones of low stress);
- Stress controlled failure through intact rock;
- A combination of structural and stress controlled failure of the bridge of intact rock between structure.

2.1 Modified NGI Rock Mass Rating (Q')

The rock mass quality index Q is modified by setting the stress reduction factor (SRF) to 1. All the other factors are unchanged. Data on the rock mass quality is collected during the normal course of underground mapping.

2.2 Rock Stress Factor (A)

The rock stress factor (A) replaces the stress reduction factor to more accurately reflect stresses acting on exposed surfaces of open stopes at depth. The factor is the ratio of intact rock strength to induced stress where:

(a) intact rock strength is defined as the unconfined uniaxial compressive strength, and

(b) induced stress is defined as the stress acting parallel to the exposed stope wall or roof under analysis.

Bridging type failure between fractures should not occur when the ratio of intact rock strength to induced stress exceeds 10. Any failure under these conditions should be related to movement on defined structure only and for this case, the rock stress factor (A) is set to 1.0.

Failure will be stress controlled as the ratio of intact rock strength to induced stress approaches unity. For this case, the rock stress factor (A) approaches zero.

Combined stress and structurally controlled failure can occur between these extremes and a straightline relationship has been assumed to determine the factor A as indicated in Figure 1. From experience, the ratio of intact rock strength to induced stress should not be less than 2. This relationship can be used to flag potential problems.

Calculated maximum concentrations of stress bounding single, rectangular openings are given in Figures 2 and 3 for increasing excavation size ratios. Induced stresses at the top of a rectangular opening can be estimated by referring to Figure 2 and in the wall by referring to Figure 3.

It will be noted that tensile stresses can be induced in the walls of tall openings with increasing values of the stress ratio (K). When these conditions are encountered, A is set to zero, indicating that block fall out by gravity will be the condition of failure.

The levels of stress induced in pillars between multiple openings are related to the virgin stress, geometry of the openings and the width of pillars between openings (both horizontal and vertical). If adjacent orebodies are being mined, the spacing between openings in the orebodies must also be considered.

Induced stresses in pillars tend to increase as:

(a) the depth of working increases;
(b) the height to span ratio of openings increases;
(c) the width of pillars decrease;
(d) the spacing between orebodies decreases.

Induced stresses are best estimated with numerical analysis techniques, using measured data on virgin stress.

Referring to Figure 2, the ratio (σ_I:σ_v) is analogous to a stress concentration factor. With openings having a height to span ratio of 4 or more, stress concentrations of the order of 4 can be expected in the backs of single openings, resulting in "high" induced stresses at depths exceeding 1,000 m. Due to the superposition of stress, concentrations of 6-8 are possible in thin pillars between stopes.

The recommended design procedure is as follows:

(a) Assess the stable dimensions of single openings using the empirical procedures outlined in this appendix;

(b) Prepare an acceptable mining layout using the assessed dimensions of stable stopes;

(c) Estimate stresses in pillars using numerical analysis techniques;

(d) Redesign pillars if stresses in the pillars exceed an arbitrary level defined by $\sigma_c/\sigma_I < 2.0$.

2.3 Rock Defect Orientation Factor (B)

This factor is included to account for the presence of persistent structure paralleling or intersecting exposed surfaces (Figure 4). The value of the factor is 0.5 for structure paralleling the exposed surface and 1.0 for structure intersecting the exposed surface at right angles. The factor is selected as follows:

(a) Determine the orientation of the most persistent set of fractures based on relative spacing and continuity;

(b) Determine the angle of intersection with the exposed surface under consideration;

(c) Refer to Figure 4 and select the orientation factor (B). Stereographic plots can be used to determine the relative angle between joint sets and design surfaces and it is recommended that they be used.

When selecting the orientation factor (B), it should be noted that the overall stability of the excavation is under assessment; some dilution in the form of block fallout or spalling can be accepted.

2.4 Orientation of Design Surface Factor (C)

Stope backs or roofs are inherently less stable than sidewalls because of the influence of gravity. Barton et al (1974) suggest that rock quality in a wall is hypothetically improved 5 times compared to a roof. They also recommend an Excavation Support Ratio (ESR) of 1.6 for permanent mine openings. However, some block fallout and spalling can be tolerated in a "non-entry" type of excavation. Therefore, it seems conservative to suggest that a vertical wall should be at least 5 x 1.6 = 8 times as stable as a horizontal roof. Referring to Figure 5, if the factor (C) for a horizontal surface is set to 1, then its value will increase as the angle of dip increases. The maximum value is 8 when the surface is vertical. The formula used to obtain the relationship between factor (C) and the angle of dip is given below and graphed in Figure 5.

Factor C = 8 - 7 cosine (angle of dip)

The factor accounts for the effect of gravity and should be applied to roofs, hangingwalls and possibly steep footwalls when adverse structure has been identified.

2.5 Shape Factor (S)

As mentioned earlier, the exposed surfaces of open excavations can be regarded as two-way spans. A shape factor can be defined as the ratio of design surface area to design surface perimeter. This ratio is defined as the "hydraulic radius" by Laubscher and Taylor (1976). The relationship of shape factor (S) to span is given in Figure 6. It should be noted that as the ratio of spans increases beyond 4:1, the factor remains relatively constant and reflects one-way spanning situations.

2.6 Graph of Stability Number (N) vs. Shape Factor (S)

It is postulated that the stability of exposed surfaces in open stopes can be assessed empirically by plotting stability number (N) versus the shape factor (S). Thus, the stability of the exposed surface decreases as the shape factor (S) increases to the point of caving.

Graphs of stability number (N) versus shape factor (S) have been plotted using data collected from case studies and from the literature, Figure 7 and 8. Figure 7 presents Golders' original data. Figure 8 presents a modified graph constructed by Potvin et al (1988). The stability number (N) has been plotted on a log scale in keeping with the presentation made by Barton et al (1974).

The divisions between stable, unstable and caving zones are defined below.

(a) Stable

The excavation will stand unsupported with occasional localized ground support to control slabbing.

(b) Unstable

The excavation will experience some localized caving, but will tend to form a stable arch. Open stoping is feasible if localized caving can be prevented by modifying extraction sequence, installing cable bolts, etc.

(c) Caving

The excavation will cave and will not stabilize until the void is full.

3.0 EXAMPLE ON THE USE OF THE STABILITY GRAPH (FROM GOLDER ASSOCIATES 1981)

An open stope is to be developed at a depth of 1,000 m. The orebody is 25 m wide and dips at 80 degrees. It is desired that the stope length be a minimum of 30 m and the height a minimum of 75 m to permit the use of in-the-hole drilling equipment. The unconfined compressive strength of the intact rock averages 120 MPa, additional data on geotechnical parameters are given in the test.

3.1 Modified NGI Rock Mass Rating (Q')

The rock mass quality data for the stope is summarized below:

	Item	Description	Value
(1)	Rock Quality	Good	RQD = 85%
(2)	Joint sets	One joint set and random	= 3
(3)	Joint roughness	Rough or irregular undulating	= 3
(4)	Joint alteration	Unaltered with surface staining	= 1
(5)	Joint water	Dry with minor surface staining	= 1
(6)	Stress reduction	Single weakness zones containing clay	SRF = 2.5

$$\text{Hence} \quad Q = \frac{85}{3} \times \frac{3}{1} \times \frac{1}{2.5} = 34$$

Determine the modified NGI Rock Mass Rating (Q') by setting SRF = 1, hence Q' = 85.

3.2 Rock Stress Factor (A)

Virgin stress has not been measured, hence the values must be estimated. The vertical stress (σ_V) is estimated at 27 MPa for a depth of 1,000 m. The K ratio of average horizontal stress (σ_H) to the vertical stress (σ_V) is estimated to be 1.375, therefore $\sigma_H = 37.1$ MPa.

Considering the values of virgin stress in the horizontal plane, assume that $\sigma_{H1} = \sigma_{H2} = \sigma_H =$ 37.1 MPa where σ_{H1} is the virgin stress parallel to strike and σ_{H2} is the virgin stress normal to strike.

Referring to Figure 2, the first step is to calculate the induced stresses in the beck (top of vertical plane) and the strike end (end of horizontal plane) of the stope.

(a) Top of Mid Stope Vertical Plane

$\sigma_V = 27$ MPa
$\sigma_{H2} = 37.1$ MPa
$K = \sigma_{H2}/\sigma_V = 1.4$

For a height to span ration of 3 and K value of 1.4, the $\sigma_I = \sigma_V$ is estimated at 2.6. Hence $\sigma_I = 2.6$ x 27 MPa = 70 MPa. Referring to Figure 1, the value of $\sigma_c = \sigma_I = 120:70 = 1.7$. As this ratio is less than 2, the back is likely to be on the verge of instability. The height of the stope should be reduced but for the purpose of this example, assume a rock stress factor (A) = 0.1.

(b) Strike End of Mid Stope Horizontal Plane

$\sigma_{H1} = 37.1$ MPa
$\sigma_H = 37.1$ MPa
$K = \sigma_{H2}/\sigma_{H1} = 1$

For a length to span ratio of 1.2 and K value of 1, then $(?)_{H1}$ is estimated at 1.0. Hence $\sigma_1 = 1.0$ x 37.1 MPa = 37.1 MPa. Referring to Figure 1, the value of $\sigma_C : s_1 = 120:37 = 3.2$ and the rock stress factor (A) = 0.25.

Referring to Figure 3, the next step is to calculate the induced stresses in the hangingwall and footwall considering the vertical and horizontal mid-stope planes. The lowest value of the estimated rock stress factor (A) is used.

(c) Mid-Stope Vertical Plane (H/W and F/W)

$K = \sigma_{H2} : \sigma_V = 37.1:27 = 1.4$

For a height to span ratio of 3 and K value of 1.4, then $\sigma_1 : \sigma_V$ is estimated at -0.1. As the value is negative, it is set to zero and σI is zero. Referring to Figure 1, the value of $\sigma_c : \sigma_1$ is greater than 10, hence the rock stress factor (A) = 1. It should be noted, however, that horizontal joints intersecting the hangingwall will open as the induced stress at the center of the hangingwall span is tensile.

(d) Mid-Stope Horizontal Plan (H/W and F/W)

$K = \sigma_{H2} : \sigma_{H1} = 37.1:37.1 = 1$

For a length to span ratio of 1.2 and K value of 1, then $\sigma_I : \sigma_{H1}$ is estimated at 0.75. Hence, $\sigma_I =$ 0.75 x 37.1 MPa = 27.8 MPa. Referring to Figure 1, the value of $\sigma_C : \sigma_I = 120:27.8 = 4.3$ and the rock stress factor (A) is 0.35.

It should be noted that both the hangingwall and footwall are in compression in the direction of strike, and tensile near the mid span) in the direction of dip.

Summarizing, the rock stress factors (A) to be used are listed below:

	Value of A
Back	0.1
H/W	0.35
F/W	0.35
Vertical End	0.25

3.3 Rock Defect orientation Factor (B)

The principal joint set is flat dipping and joints are closely spaced in the range of 7 - 15 cm apart. joint surfaces are unaltered with surface staining.

The orebody is 25 m wide and dips at 80 degrees. Referring to Figure 4, the orientation factor for the exposed surfaces are as follows:

Exposed Surface	Orientation (degrees)	Value of (B)
Back	0	0.5
Hangingwall	100	1.0
Footwall	80	1.0
Vertical End	90	1.0

3.4 Design Surface Orientation Factor (C)

The hangingwall and footwall dip at 80 degrees, hence surface orientation factors for the exposed surfaces are as follows from Figure 5.

Exposed Surface	Inclination (degrees)	Value
Back	Horizontal	1
Hangingwall	80	6.5
Footwall	+90	8
Vertical End	90	8

3.5 Values of Stability Numbers (N)

Values of N = Q' x A x B x C for the exposed surfaces are summarized below:

Exposed Surface	Value of (N)
Back	4.3
Hangingwall	220.0
Footwall	240.0
Vertical End	170.0

3.6 Values of Shape Factor (S)

Values of (S) equal to the ratio of the area of t to the perimeter of the exposed surface are given below.

Exposed Surface	Value of (S)
Back	6.8
Hangingwall	10.7
Footwall	10.7
Vertical End	9.4

3.7 Comments on Analysis

All of the walls plot in the stable area of the stability number versus shape factor graph; however, the roof plots in the zone between "unstable" and "caving". If these results are accepted, then the height of the stope should be reduced to lessen roof stress.

Stresses are estimated for the central sections of the spans, but corner stresses will be much higher. Spalling can be anticipated in the upper corners along strike and cable bolting will not prevent this. However, cable bolting would be effective for general roof support and in areas that have spalled to a stable shape.

Localized sloughing can be expected from the central portion of the hangingwall (and footwall) but the vertical ends should be sound. However, secondary joint sets are randomly orientated; hence, some random spalling and sloughing can be anticipated.

The analysis was for a single opening and it can be appreciated that numerical analysis techniques are necessary to obtain a reasonable picture of stress distribution resulting from the interaction of multiple openings. The roof of the single stope analyzed may have spalled to a stable shape to permit support by cable bolts. However, if the roof was the bottom of a horizontal pillar separating a worked out stope above, then it probably would have failed completely. Failure would have resulted from the superposition or the interaction of stresses between the two openings to give much higher values. The effect of superposition of stresses increases as the thickness of the pillar decreases as illustrated by Hoek & Brown (1980). The horizontal pillar can only be stabilized by increasing pillar thickness or decreasing stope height; however, these measures may not be economic. The vertical end is quite stable; hence, vertical pillars separating stopes on strike should also be stable if of reasonable length.

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Golder Associates, 1981. "Prediction of Stable Excavation Spans for Mining Depths Below 1000 Meters in hard Rock". Report to CANMET, DSS Serial No. 05480-0081.

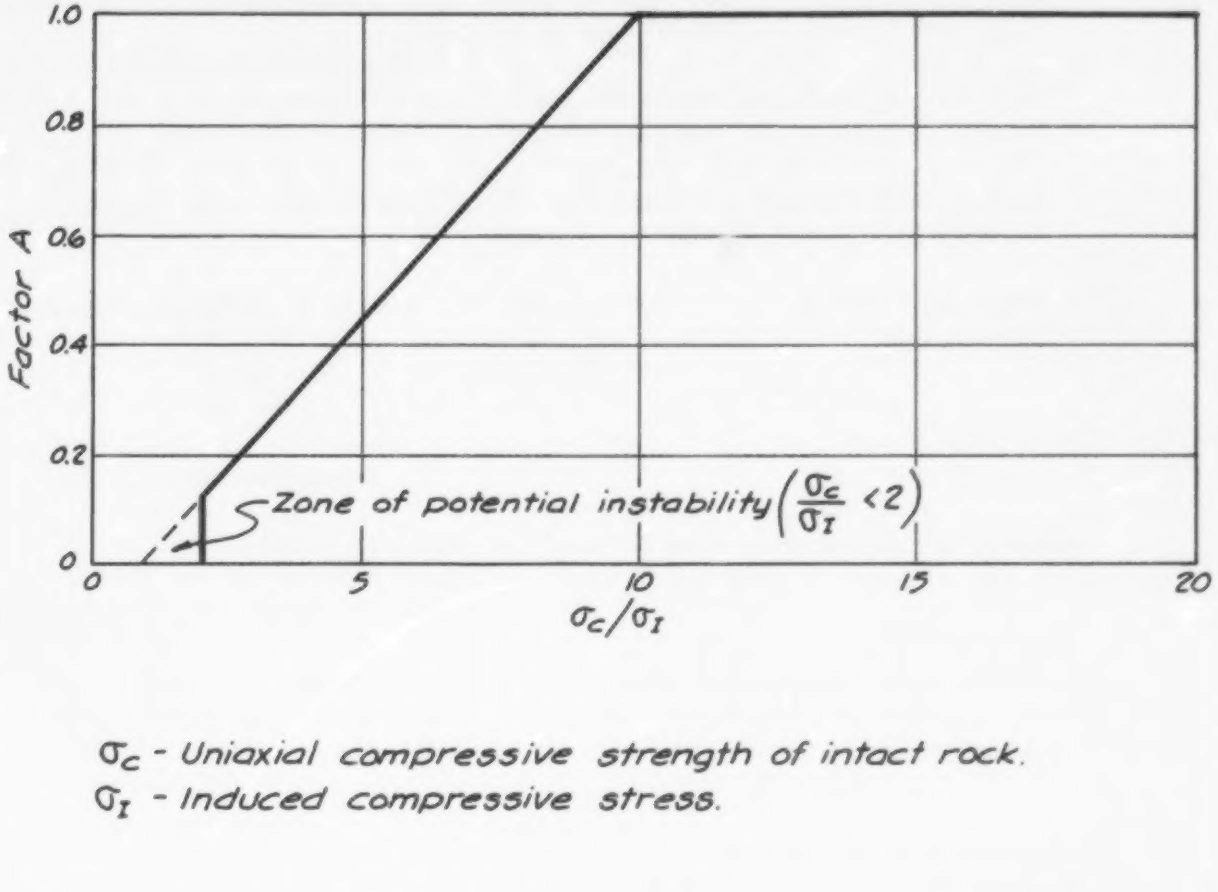
Barton, N., Lien, R. and Lunde, J., 1974. "Engineering Design to Tunnel Support", Rock Mech. Vol. 6, No. 4.

Potvin, Y., Hudyma, M. and Miller, H.D.S., 1988. "The Stability Graph Method of Open Stope Design", Presented at 90th CIM AGM, Edmonton.

Hoek, E. and Brown, E.T., 1980. "Underground Excavations in Rock", Institute of Mining and Metallurgy, London, England.

GRAPH TO DETERMINE THE ROCK STRESS FACTOR (A)

Figure **1**



σ_c - *Uniaxial compressive strength of intact rock.*

σ_I - *Induced compressive stress.*

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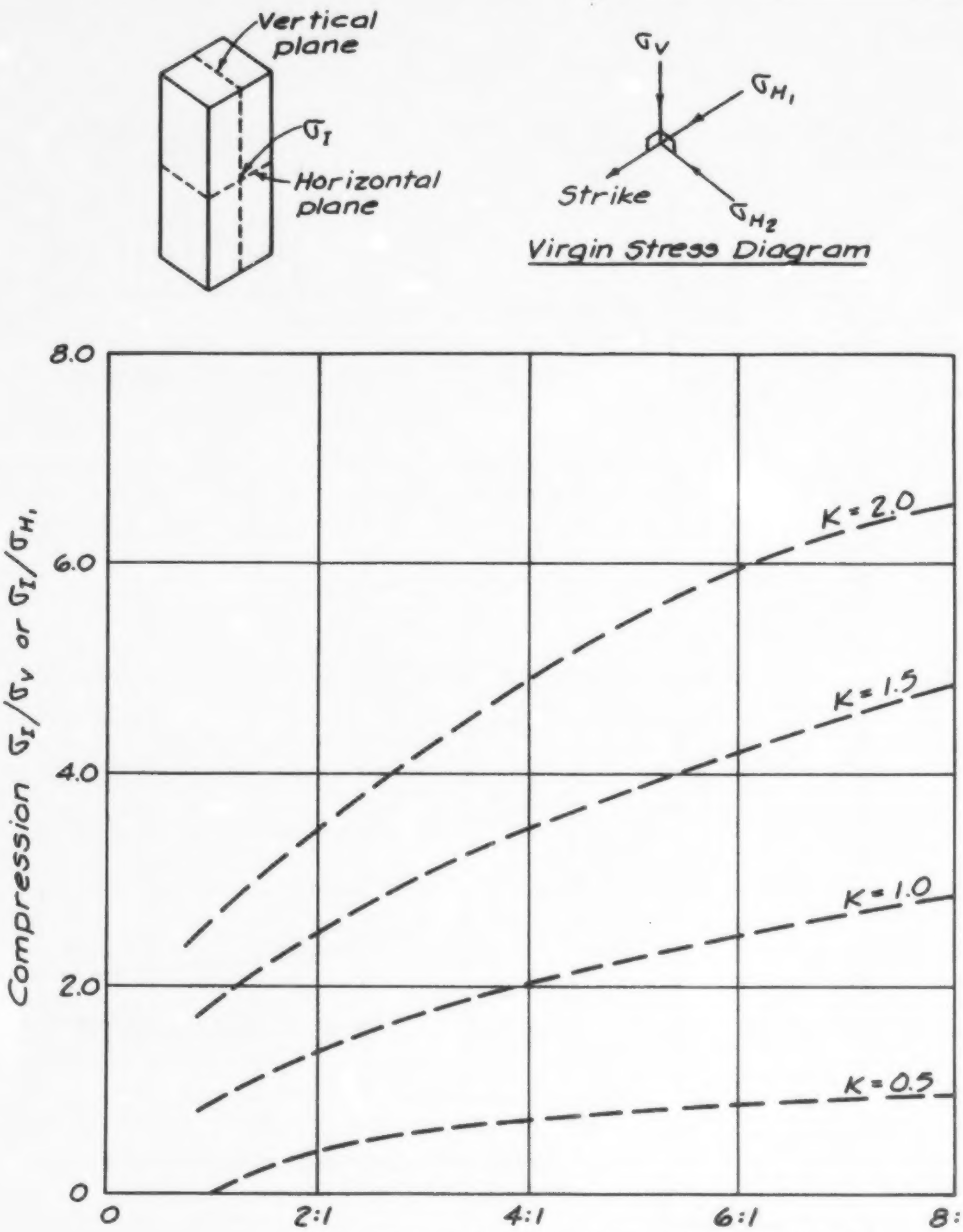
GRAPH TO DETERMINE INDUCED STRESSES IN BACKS AND ENDS OF SINGLE OPENINGS

Figure **2**



LEGEND

σ_I = Induced stress

σ_V = Vertical virgin stress

σ_{H_1} = Horizontal virgin stress on strike

σ_{H_2} = Horizontal virgin stress normal to strike

Horizontal plane, $K = \sigma_{H_2}/\sigma_{H_1}$

Vertical plane, $K = \sigma_{H_2}/\sigma_V$

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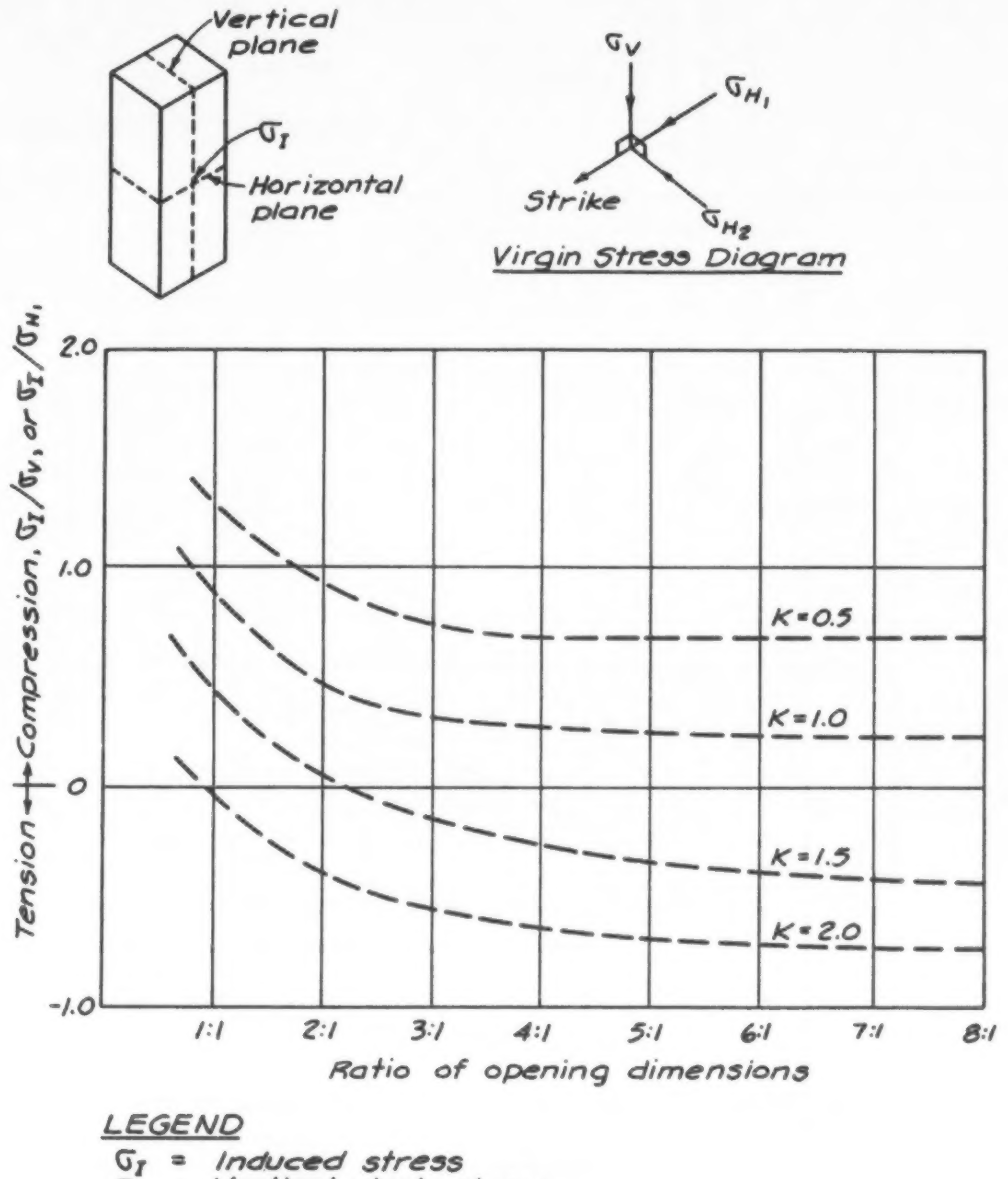
GRAPH TO DETERMINE INDUCED STRESSES IN HANGINGWALLS OF SINGLE OPENINGS

Figure 3



LEGEND

σ_I = Induced stress
σ_V = Vertical virgin stress
σ_{H_1} = Horizontal virgin stress on strike
σ_{H_2} = Horizontal virgin stress normal to strike
Horizontal plane, $K = \sigma_{H_2}/\sigma_{H_1}$
Vertical plane, $K = \sigma_{H_2}/\sigma_V$

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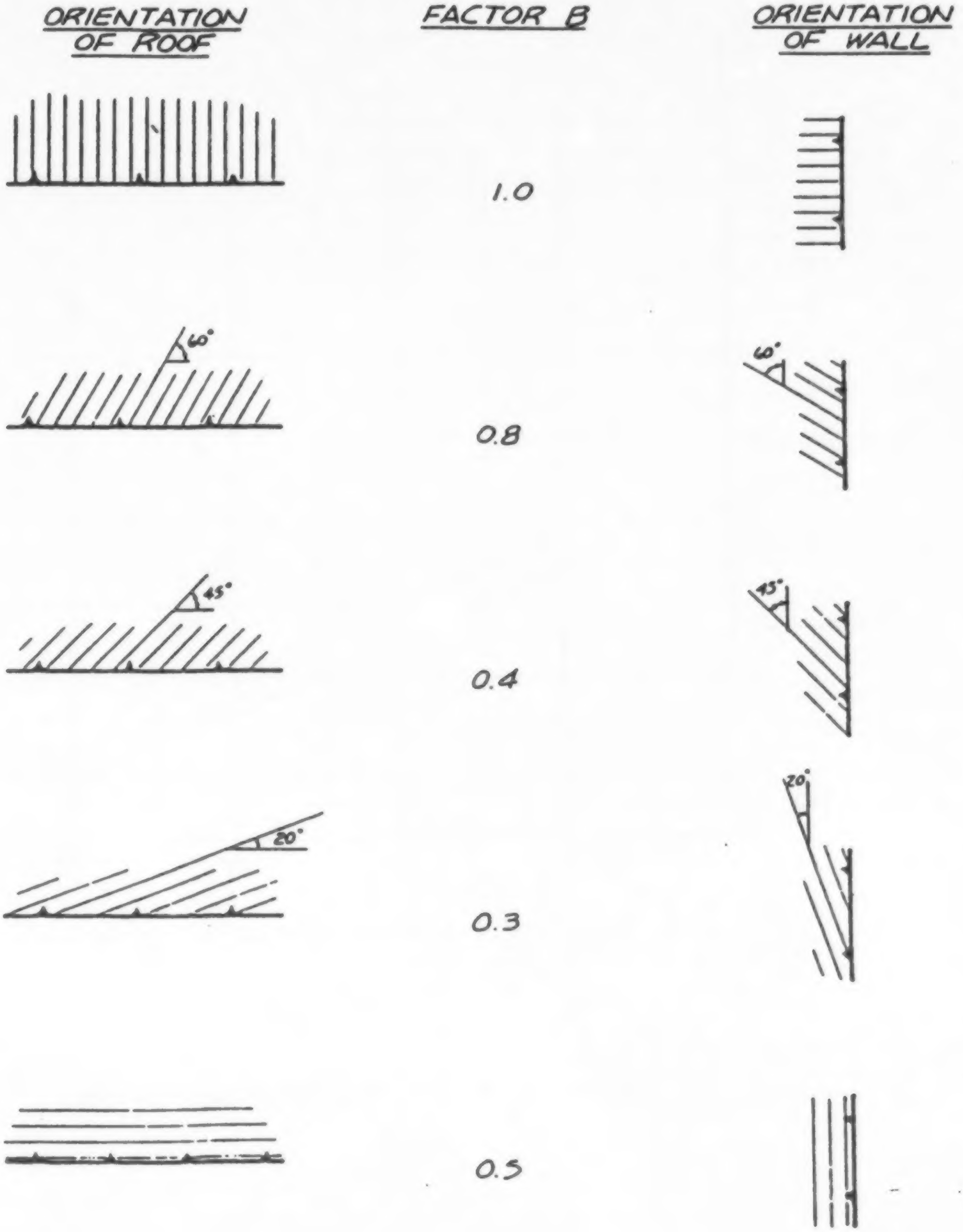
ORIENTATION ADJUSTMENT

Figure **4**


ORIENTATION OF ROOF
FACTOR B
ORIENTATION OF WALL
1.0
60°
0.8
60°
45°
0.4
45°
20°
0.3
20°
0.5

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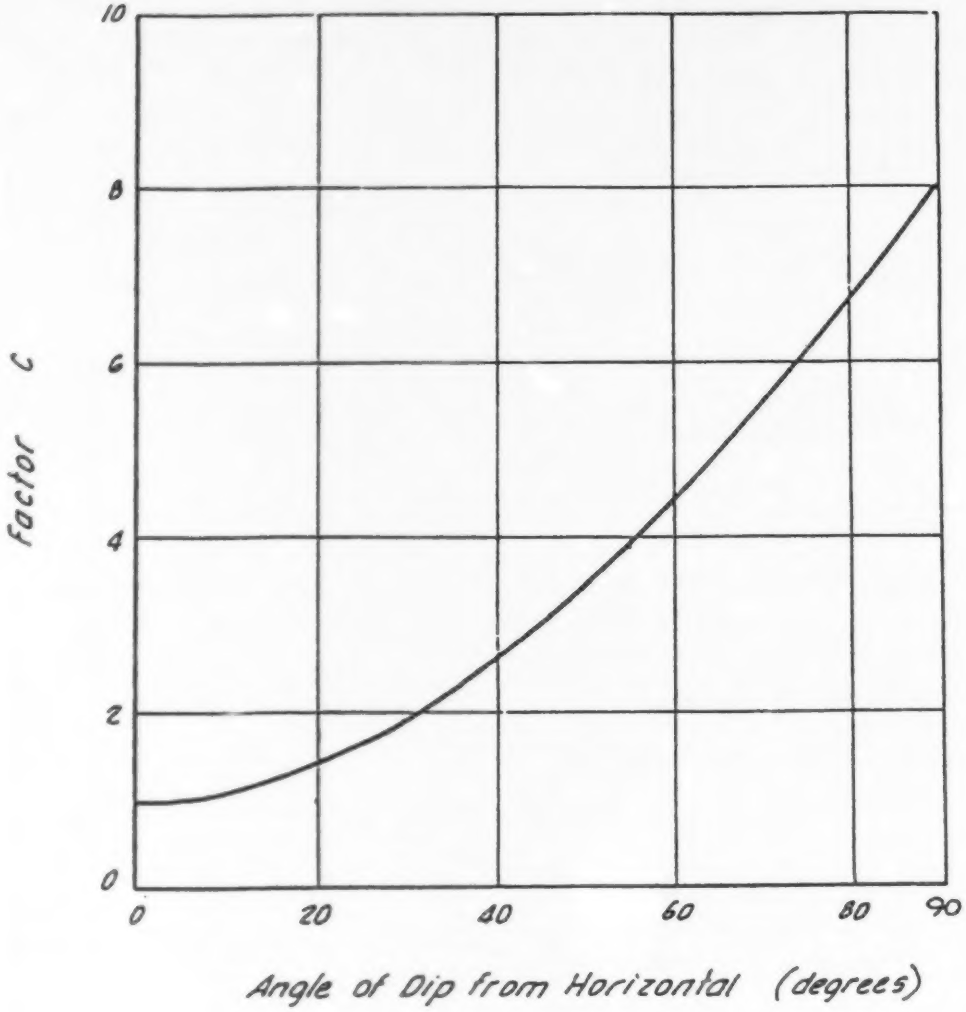
DESIGN SURFACE ORIENTATION FACTOR (C)

Figure **5**



Factor C = 8 - 7 Cosine (angle of dip)

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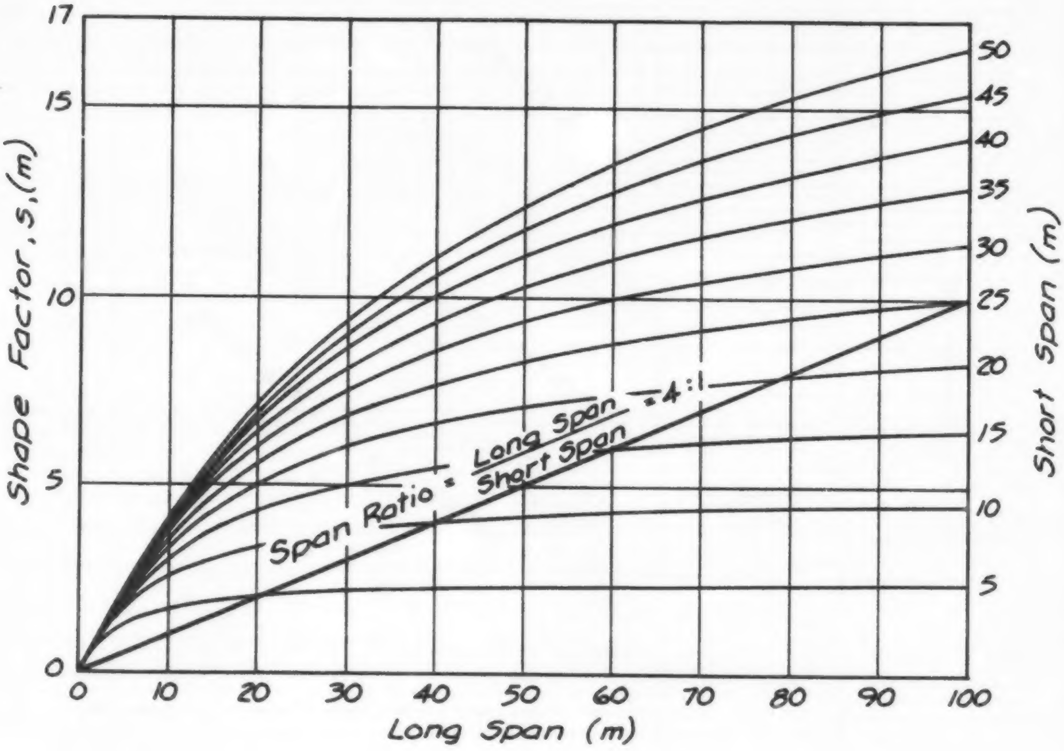
SHAPE FACTOR VS. SPANS

Figure **6**



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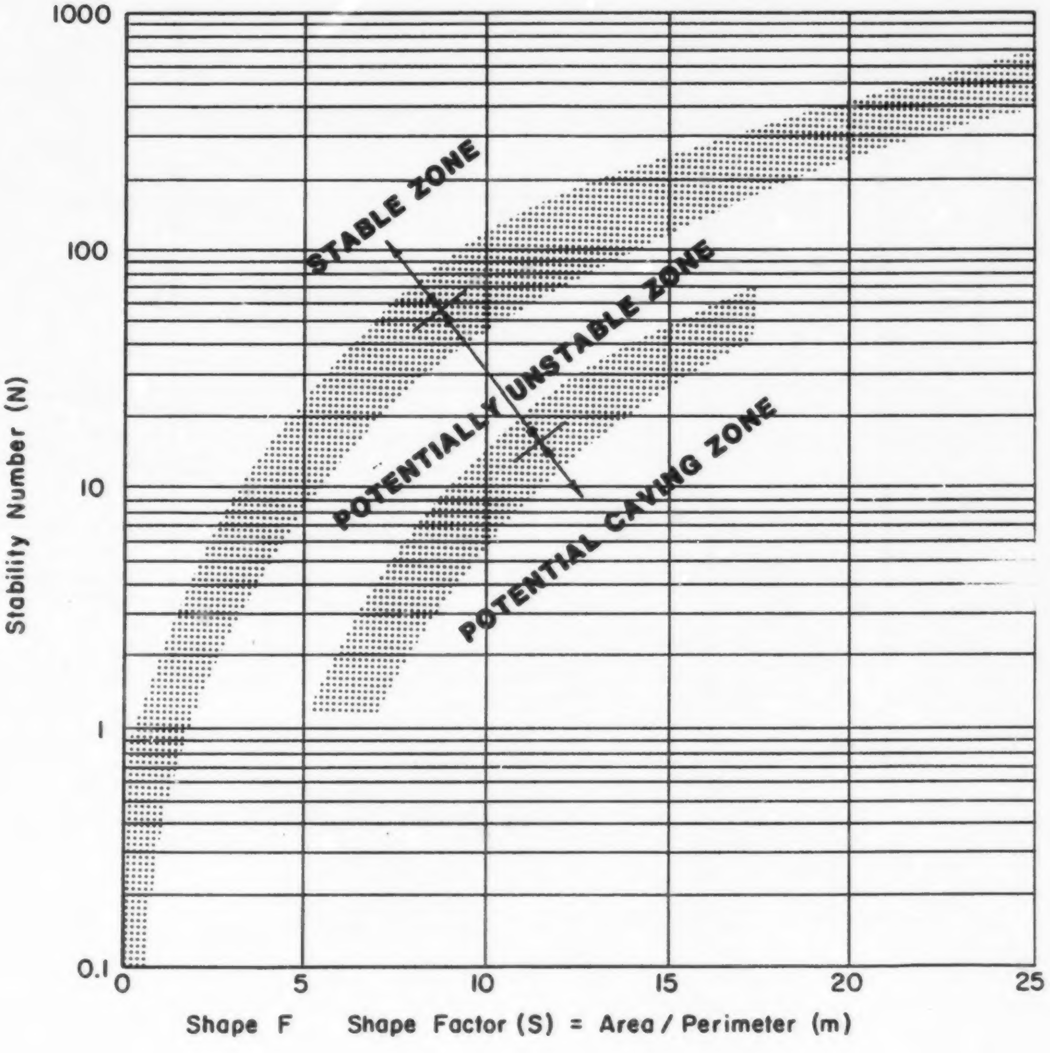
GRAPH OF STABILITY NUMBER (N) VERSUS SHAPE FACTOR (S)

Figure **7**



(after Mathews et al 1981)

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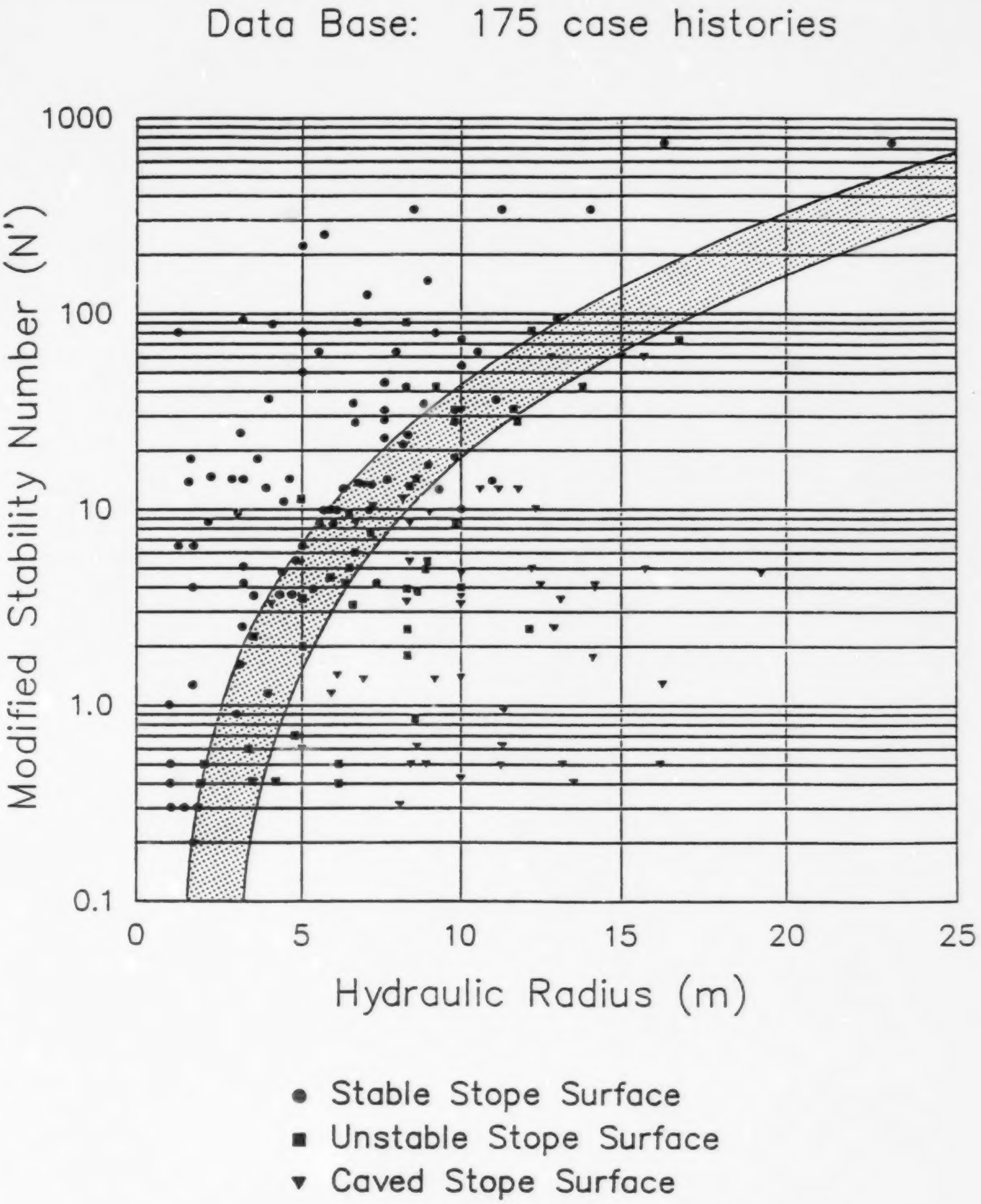
REVISED STABILITY GRAPH

Figure **8**



(after Potvin, 1988)

PROJECT NO. 882-1612 DRAWN R.Q. REVIEWED DATE Aug '88